

SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CALIFORNIA

SUB-APPENDIX B.2 – COASTAL LIFE SAFETY RISK

JULY 2026

USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO



**US Army Corps
of Engineers**



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Attachment 1: SQRA Report

Acronyms and Abbreviations

Acronym	Definition
CSRM	Coastal Storm Risk Management
FWOP	Future Without Project
FWP	Future With Project
MHHW	Mean Higher High Water
NWS	National Weather Service
PDT	Project Delivery Team
RSLC	Relative Sea Level Change
USACE	U.S. Army Corps of Engineers

1.0 General Methodology

USACE PB 2019-04 provides guidance for incorporating Life Safety into Coastal Storm Risk Management (CSRM) Studies. This guidance is based on levees and the San Francisco CSRM essentially functions as a levee at higher rates of RSLC when the flood protection structures are loaded more frequently (or constantly) than they are during rare storm events. Paragraph 4b requires sources of life risk to be identified, while evaluating the increase, or transformation of risk to life due to sea level rise and protection provided by the CSRM. Paragraph 5d requires a life safety risk assessment to be conducted for all new levee systems. The Project Delivery Team (PDT) conducted semi-qualitative risk assessment which is attached. For the initial alternatives array, the PDT conducted an assessment for coastal life safety which is documented in this supplemental report.

The metrics documented in this supplemental report are qualitative and are intended to provide a high level, relative comparison of performance between the future without project (FWOP) condition and the various project alternatives. Evaluations considered two CSRM failure modes and two water level scenarios. As such, the metric scores are provided on a scale of 1 to 5, with the score of 1 indicating a poor outcome and 5 indicating a positive outcome with respect to the metric being evaluated. The information used to reach these scores is based upon professional judgment within the PDT. The specific methodology for each metric and generalized rubric for scoring for each is provided in the sections below.

2.0 Life-Safety Performance Relative to Coastal Storm Risk

This metric is intended to qualitatively compare the performance of the alternatives with the life-safety hazard caused by coastal water levels that are unabated by the CSRM system, and/or exacerbated by a failure of the CSRM system when resisting coastal waters (i.e., fragility). The evaluation of the life-safety hazard is dependent on the coastal water levels and the frequency at which they are expected to be present. Safety is considered compromised when an unexpected failure of the CSRM system results in more than 2 feet of water suddenly entering urban areas of the city without warning.

3.0 Future without Project Condition

In the FWOP condition, the life-safety hazard associated with inundation of coastal water is expected to be low, due to the long and slow development of the coastal hazard. Tides rise and fall daily, and extreme storm events coinciding with high tides will lead to exceptionally high-water levels. However, these tidal fluctuations do not occur suddenly, which allows time for most of the population to safely evacuate without presenting a life safety hazard. In addition to the higher water levels, storm events may also bring significant wave energy to the shoreline, causing waves to overtop structures, as was experienced during the Winter 2023 storms. These waves may present a low to moderate hazard, for members of the public that do not heed warnings.

4.0 Future with Project Condition

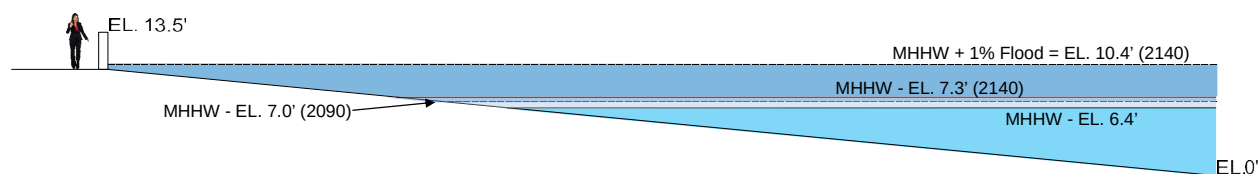
For the future with project (FWP) condition, the life-safety hazard relates to each relative sea level change (RSLC) curve considering the following conditions:

- CSRM Overtopped with water level equal to the mean higher high water (MHHW) plus the 1% storm surge.
- CSRM Overtopped with water level equal to the MHHW (i.e., Sunny Day Failure).
- CSRM sudden failure with water level equal to the MHHW plus the 1% storm surge.
- CSRM sudden failure with water level equal to the MHHW.
- Failure of the CSRM system due to an earthquake, while loaded with the typically expected water level (*Note: Not addressed in this analysis)

The 1-5 rating scale for the anticipated outcome is as follows: 1=Poor, 2=Moderately Poor, 3=Neutral, 4=Moderately Positive, 5=Positive. In the absence of performing a risk assessment for each alternative, the following assumptions were made when assessing the performance of each condition above. For overtopping, each flood defense measure is assumed to not fail during an overtopping event. Sufficient advance warning would be provided to residents as several gages are along the waterfront and the National Weather Service (NWS) provides sufficient warning when large storms are forecasted in the bay area. A Sunny Day Failure is based on normal loading conditions, therefore minimal warning time would be provided as structures are not expected to fail in these circumstances. Depending on the RSLC curve adopted for design and the amount of sea level rise at the time of failure, the life safety risk due to flooding can range between positive and poor outcomes. The figures below represent the forecasted elevations based on the USACE Low, Intermediate, and High curves.

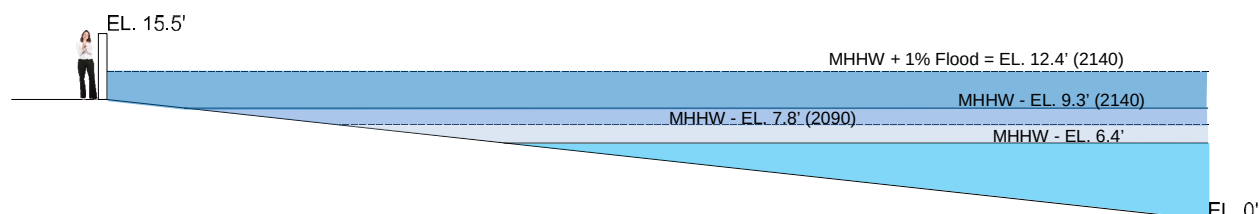
5.0 USACE Low RSLC Curve

For the USACE Low RSLC curve, which is estimated to be +0.9 feet at 2140, the MHHW plus RSLC is at elevation 7.3 feet (representing the tide-only sunny day scenario), and at 10.4 feet when including a 1% (100-year) storm surge and RSLC. A sunny day failure would result in no additional flooding to the study areas since a majority of the waterfront shoreline is between elevations 10.0 feet and 12.0 feet, with the lowest elevation down to elevation +8.5 feet. Stillwater levels associated with the low RSLC curve in 2140 and storm surge would also only slightly exceed the existing shoreline elevations in portions of the project length and touch the proposed CSRM measures based on an assumed low RSLC curve. If a failure of an isolated area of a CSRM measure did occur, minimal flooding depths would be seen, approximately equal to what currently occurs today, with most areas experiencing no flooding. Life loss associated with failure of any proposed CSRM measure with these potentially retained water levels are judged to be zero in the USACE Low RSLC curve scenario.



6.0 USACE Intermediate RSLC Curve

The USACE Intermediate RSLC curve is forecasted to raise sea level by 1.4 feet in 2090 and 2.9 feet by 2140. This results in MHHW elevations of 7.8 feet (2090) and 9.3 feet (2140). On a sunny day (tide only in 2140), none of the proposed CSRM measures would be loaded, thus resulting in no flooding in the event of failure. Including the 1% storm surge in 2140, increases the water elevation to 12.4 feet, thus loading the proposed CSRM measures with approximately 0.4 feet to 2.4 feet depth of water. A failure of the flood defenses with water levels at elevation 12.4 feet would result in somewhat continuous flooding with water depths of 2 feet or less across most of the study area. Life loss associated with failure of any proposed CSRM measure with the USACE Intermediate RSLC curve and associated retained water levels are judged to be minimal, since water depths are less than 2 feet on average.

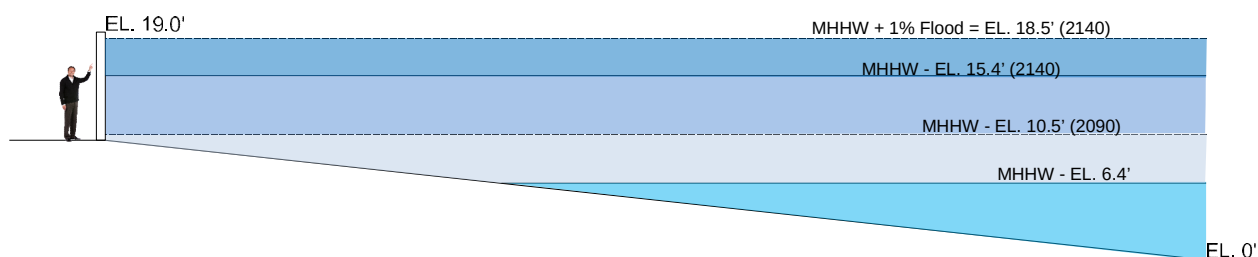


7.0 USACE High RSLC Curve

The USACE High RSLC curve is forecasted to raise sea levels by 4.1 feet in 2090 and 9 feet in 2140. This results in MHHW elevations of 10.5 feet (2090) and 15.4 feet (2140) for the sunny day (tide only) case. No matter which structural CSRM measures are implemented (i.e., for USACE Low, Intermediate, High RSLC curves) there will be minimal to no flooding in the sunny day event under the USACE High RSLC curve by 2090. The sunny day MHHW elevation is forecasted to be 15.4 feet in 2140, thus requiring CSRM measures to be built to at least elevation 15.5 feet resulting in water depths at the structure between approximately 2.5 feet to 5.5 feet during a higher high tide event. A failure of the flood defense in this scenario would result in widespread flooding and potentially significant loss of life. Minimal to no warning would be provided as the failure would not be related to a forecasted storm event and the local population would not expect a failure to occur.

Assuming rational and responsive behavior of businesses and residences protected by the CSRM system, measures built to an elevation of 13.5 feet would result in fewer lives

lost since the 2140 sunny day water elevation would be known to overtop this line of protection and action would be taken to ensure adequate warning is provided to the public to warn against anticipated flooding. Adding the 1% storm surge in 2140, produces an elevation of 18.5 feet, thus requiring proposed CSRM measures to be built to an elevation of 19.0 feet with approximately 6.5 feet to 8.5 feet of retained water depth if USACE High RSLC and the 1% storm event occurs. A failure with the bay water level at this elevation would result in catastrophic, widespread flooding and significant life loss occurring. Life loss potential would be significantly less and potentially zero for CSRM measures built to elevations of 13.5 feet and 15.5 feet as the 2140 MHHW plus 1% storm surge water elevation would be known to overtop these lines of protection and adequate warning could be provided to the public to warn against anticipated flooding.



For this qualitative assessment the alternatives were determined to score as follows for the failure conditions and sea level change scenarios as follows:

Overtopping with 2140 MHHW+1% storm surge (note that overtopping is only anticipated for Alts C & D with the High RSLC curve)

Alt	Low RSLC (EI 10.4')	Int RSLC (EI 12.4')	High RSLC (EI 18.5')
A	NA	NA	NA
B	5	4	2
C (13.5')	5	5	4
D (15.5')	5	5	4
E (19.0')	5	5	5
F (19.0')	5	5	5
G (19.0')	5	5	5

Sudden Failure with 2140 MHHW+1% storm surge (* indicates Alt already overtopped)

Alt	Low RSLC (EI 10.4')	Int RSLC (EI 12.4')	High RSLC (EI 18.5')
A	NA	NA	NA
B	5	3	2
C (13.5')	5	5	NA*
D (15.5')	5	5	NA*
E (19.0')	5	5	1
F (19.0')	5	5	1
G (19.0')	5	5	1

Overtopping with 2140 MHHW, Sunny Day
(note that overtopping is only anticipated for Alt C with the High RSLC curve)

Alt	Low RSLC (EI 7.3')	Int RSLC (EI 9.3')	High RSLC (EI 15.4')
A	NA	NA	NA
B	5	4	2
C (13.5')	5	5	4
D (15.5')	5	5	5
E (19.0')	5	5	5
F (19.0')	5	5	5
G (19.0')	5	5	5

Sudden Failure with 2140 MHHW, Sunny Day
(* indicates Alt already overtopped)

Alt	Low RSLC (EI 7.3')	Int RSLC (EI 9.3')	High RSLC (EI 15.4')
A	NA	NA	NA
B	5	3	2
C (13.5')	5	5	NA*
D (15.5')	5	5	2
E (19.0')	5	5	1
F (19.0')	5	5	1
G (19.0')	5	5	1

The PDT evaluated each alternative as a whole and did not separate the northern waterfront from the southern. The retreat, or fall back, alternatives (F and G) would result in more positive outcomes if the scores were assigned per reach as portions of the population at risk would be removed from the area, the existing ground surface generally being higher resulting in shorter CSRM measures and resulting discontinuity of back flooding potential across the project length. However, the scores reflect the single, worst-case scenario which assumes failure in one location would allow back flooding of the entire project. Additionally, a detailed risk assessment would account for the probability of failure as it relates to the type and design of the CSRM system, which was not taken into consideration for this qualitative risk assessment. As such, this coastal life safety score should be taken into consideration for plan selection but should not be used as a screening tool based upon individual alternative scoring.

San Francisco Waterfront Storm Damage Reduction System Semi-Quantitative Risk Assessment

October 1, 2024



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Controlled by: Alex Nelson, P.E., USACE Levee Safety Center
CUI Category: General Critical Infrastructure Info
Distribution/Dissemination Control: FEDCON
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US Army Corps
of Engineers®
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San Francisco Waterfront Storm Damage Risk Reduction System

Semi-Quantitative Risk Assessment

October 1, 2024

South Pacific Division

San Francisco District (Tulsa District for design)

San Francisco Waterfront Storm Damage Risk Reduction System

San Francisco Bay, California

Levee, Floodwall, Seawall, Non-structural

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EXECUTIVE SUMMARY

A Semi-Quantitative Risk Assessment (SQRA) was completed in general accordance with Engineer Circular (EC) 1165-2-218 *USACE Levee Safety Program* (USACE 2021), consisting of a facilitated Potential Failure Mode Analysis (PFMA), a site visit, and a risk assessment of potential failure modes judged to be risk drivers.

Project Description

The low-lying assets and economic activity along the San Francisco Waterfront are at risk of flooding from coastal storms and extreme high tides, and from the potential failure of the century-old San Francisco seawall, which could result from structural deterioration or earthquake induced shoreline instability (liquefaction or lateral spreading). The San Francisco Bay Area is a seismically active region, and a major earthquake could happen within the study area at any time.

The risk of overtopping in the San Francisco Waterfront is exacerbated by rising sea levels. The rate of relative sea level change (RSLC) is uncertain and could rise from 1 to 10 feet by 2140 according to some of the predictive models. By 2040 under a High SLC curve, over 500 structures will be vulnerable to flooding from the 1% annual exceedance probability (AEP) event, which is an extreme storm with a 1% chance of happening in any given year. Some of these assets include San Francisco Municipal Transportation Agency (SFMTA) facilities and track, City facilities, and commercial real estate. Under the High SLC curve, average annual damages exceed \$100 million by 2046; this is driven both by potential damages from infrequent, high water level events, but also from repetitive flooding occurring in low-lying areas. By 2090 under the High SLC curve, there could be up to three feet of sea level change. This increases both the spatial extent of infrequent storm events and the effects of frequent flooding events.

The Tentatively Selected Plan (TSP) is a cost effective, hybridized plan that combines retreat and defend measures, scaled to perform under the lowest initial risk and to adapt to risk of a higher rate of RSLC as a potential subsequent action.

Risk Characterization

The incremental risk to the Coastal Storm Risk Management system is driven by overtopping breach risk of the levee. The system features including levees, floodwalls, closures, bulkhead walls, and pier walls are expected to perform well under hazards up to design levels. Infrequent events in excess of the design conditions will overtop the system at some level, at which point it is expected that earthen levee embankments would be the most susceptible features to overtopping breach. If overtopping breach were to occur within the project design life of 50 years and the expected range of relative sea level change, it is expected that breach depths would be low, life safety risk would be low, and economic damages would be limited due to the short duration of peak tide events and the limited breach flow through a breach. The focus of this risk assessment is on the first action measures over the 50 year design life, but the team did qualitatively address concerns for second action conditions for 50-100 years in the future. In the case that defensive measures are taken rather than retreat measures, a failure of a raised levee could have higher life safety risk and economic damages due to the higher depths of flooding associated with a breach. This

incremental risk could be greater if the probability of breach prior to overtopping and breach from overtopping is not adequately reduced through design measures. Overall, the first action measures are expected to significantly reduce average annual damages for areas within the CSRM system and the team judges the proposed project life safety risk to be tolerable.

Recommendations

1. Continue with the proposed plan to reduce flood risk through a comprehensive system of structural and non-structural measures. (All PFMs; TRG 1; PDT)
2. Develop comprehensive post-earthquake inspection plan for the CSRM. (All seismic PFMs; TRG 3; Sponsor)
3. Follow risk hierarchy for closure type (gates, fill placement, etc.) in the design of any closures, with up-and-over / crossing being first, swing gates and roller gates second, passive systems third, and sandbags last. (Closure PFMs; TRG 4; PDT)
4. Include plan to install, maintain, and update as needed instrumentation to track settlement and ground motions along the project alignment. These could include surveyed monuments, seismic monitoring, or other similar instrumentation. (All PFMs; TRG 3; PDT)
5. Develop project-specific ship impact design criteria to account for appropriate impact loads across the designed system. Consider the type of structure (levee, wall, etc.), the toe elevation, exposure to ship traffic, predominant winds and currents, etc. Consider dolphins, as needed, to reduce impact load requirements.
6. Update Emergency Action Plan (EAP) and create/add a Flood Warning Emergency Evacuation Plan (FWEPP) in the TSP, which can integrate into existing City/County emergency systems. Include breach and non-breach risk to underground parking structures and underground transit. Include specific forecasting and gaging to determine when transit is shutdown and when population is required to evacuate. Pier buildings may need to be evacuated if pier walls are loaded.
7. Consider the risk of additional expected consequences for underground occupants, damages, and loss of service (parking garages, BART, etc.)
8. Develop robust O&M plan that includes plan to inspect and maintain EWN coarse beach if using it to reduce structure heights based on effectiveness to reduce waves. Also include O&M of wave breaks as they are important to wave loading of northern reaches. If EWN options are pursued and provide flood risk reduction, they need to be monitored and maintained to the same degree as the FRM features. If EWN options are pursued only for environmental reasons, they can be allowed to adapt naturally, however, they should not be considered to provide a risk reduction. For example, an Ecotone levee option should not be used to reduce the levee height if the coarse beach has some likelihood of settlement or erosion.
9. Include two means of closure on all culvert outlets as a best practice to reduce backflow risks.
10. Include valves on both sides of levee for pressurized pipes that pass beneath levee or floodwalls.
11. Include monitoring plan to collect bathymetry along the toe of the alignment to identify any concerns of prop wash erosion or overdredging.

12. Include a temporary measure to "ring" the Embarcadero Station in the event the San Francisco Transition Structure fails and floods the BART tunnel. This measure could also be used in the interim to reduce risk of flooding over the existing bulkhead wall from entering the station entrance on the landside, prior to the end of construction.

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1. Findings and Recommendations

A Semi-Quantitative Risk Assessment (SQRA) was completed in general accordance with Engineer Circular (EC) 1165-2-218 *USACE Levee Safety Program* (USACE 2021), consisting of a facilitated Potential Failure Mode Analysis (PFMA), a site visit, and a risk assessment of potential failure modes judged to be risk drivers.

1.1. Incremental Life Safety Risk

A total of seventy potential failure modes (PFM) were identified by the SQRA team for consideration. PFM 1 – Embankment Overtopping Erosion Breach – was fully evaluated by the team. The remaining PFMs were not developed in detail because they are expected to be addressed through Pre-Construction Engineering and Design (PED) and are not expected to be risk-drivers for the system. They are discussed in Appendix A.

A risk matrix has been established to portray the incremental life safety risk (due to failure) with annual probability of failure (APF) on the vertical axis and the associated incremental life loss on the horizontal axis. Cell divisions correspond to order-of-magnitude ranges of APF and incremental life loss. The risk matrix is similar to the $f-\bar{N}$ diagram for portraying societal incremental life safety risk for quantitative risk analysis.

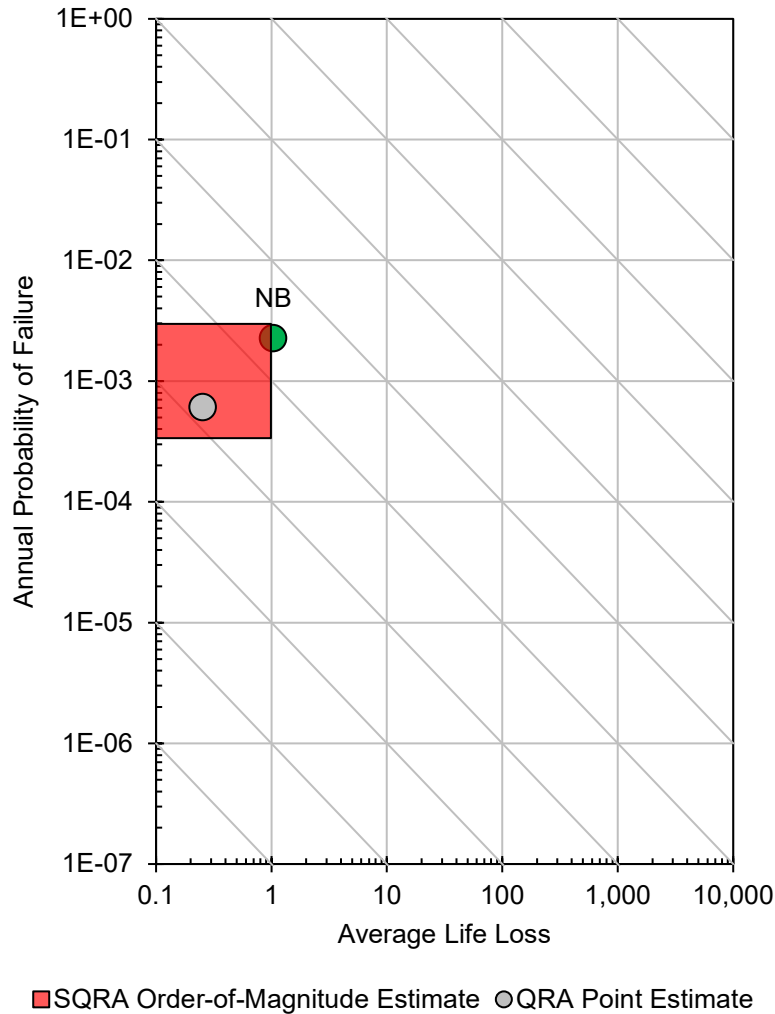


Figure 1. Societal incremental life safety risk matrix, showing incremental life safety risk (red) and non-breach life safety risk (green) for the end of the project life (year 2090)

Additional details about the risk-driving PFMs and associated incremental life loss are provided in the following sections.

1.1.1. PFM 01: Overtopping breach of the levee

If water levels, due to a combination of high tides and a storm event, are forecasted to exceed the top of the various flood risk management features, the levee embankment is expected to have a higher probability of breach compared to floodwalls and bulkhead walls. While the vegetated levees that are expected to be low in overall height are likely able to tolerate some amount of overtopping flows without significant damage or full breach, especially given the short nature of peak tide levels, there are infrequent storm events that can result in enough overtopping to fail the embankments. The risk of overtopping increases over the life of the project due to the relative sea level change from rising high tides and

potential settlement of the constructed levee grade. However, the incremental risk related to overtopping breach is expected to be low as the design includes additional height to account for high uncertainty in the future sea level change condition and additional height to account for limiting wave overtopping from potential coincident waves during peak water levels. The frequency of overtopping is expected to be less frequent than 1/1000 at the end of construction (year 2040). At the end of the project life (year 2090), the overtopping frequency is expected to be around 1/500. If a breach were to occur, the expected life loss from both the breach and a non-breach condition are expected to be close to zero. This low life loss estimate is driven by limited inundation area due to the rising terrain, the low depths of flooding, and the ability for advanced warning to take protective action. The team focused on the risk of overtopping breach during the proposed first action design, although, if a second action is pursued and sea level change follows the high projections, life loss would be greater from a breach due to the increase in flood depths of a failure of a taller levee with higher flood loading.

1.2. Incremental Economic Risk

A total of seventy potential failure modes (PFM) were identified by the SQRA team for consideration. PFM 1 – Embankment Overtopping Erosion Breach – was fully evaluated by the team. The remaining PFMs were not developed in detail because they are expected to be addressed through Pre-Construction Engineering and Design (PED) and are not expected to be risk-drivers for the system. They are discussed in Appendix A.

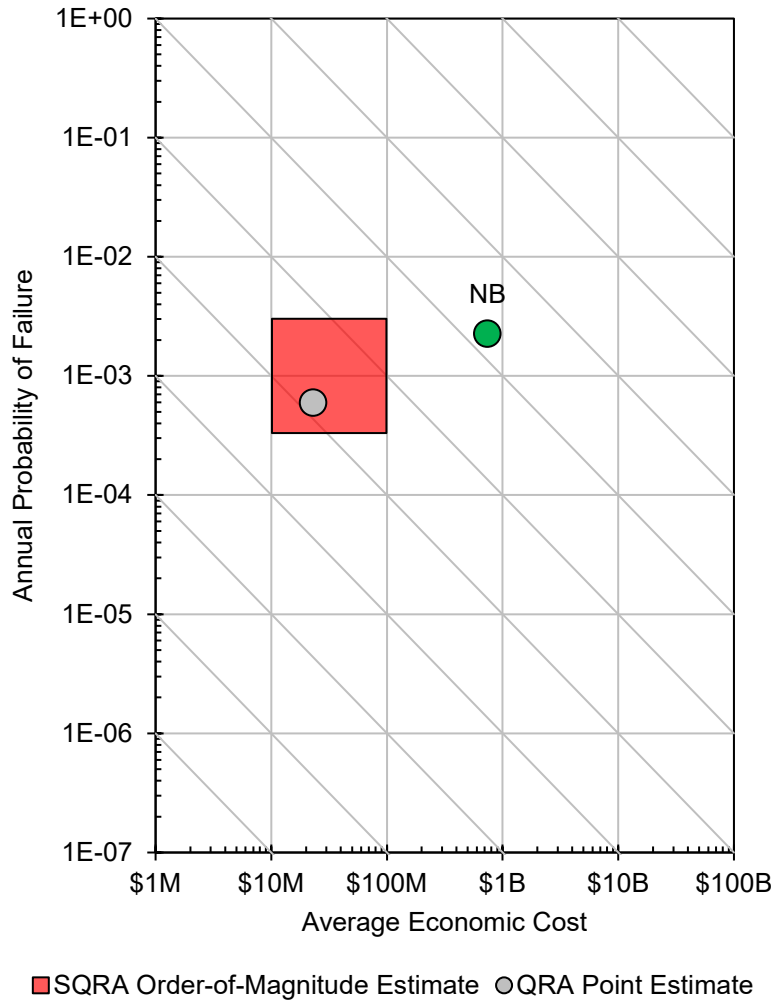


Figure 2. Incremental economic risk (red) compared to non-breach economic risk (green) for the end of project life condition (year 2090).

1.2.1. PFM 01: Overtopping breach of the levee

Similar to the life safety risk, incremental economic risk is driven by the probability of an overtopping breach of a levee feature. A breach at the 2090 design loading condition is expected to result in up to \$80 million in damages. This estimate is below the expected annual damages of hundreds of millions of dollars that was documented in the planning study, likely because a breach would limit flow to the project area especially over the short duration of peak high storm tide elevations.

1.2.2. Non-Breach Life Safety Risk

Non-breach risk occurs when the flood capacity of the system is exceeded. At this point, the system transitions from managing the flood to passing the flood. For levees, the transition occurs at the overtopping stage. This elevation corresponds to the annual probability of non-breach inundation but may not result in life loss. The annual exceedance probability (AEP) when life loss begins to occur ($AEP_{N>0}$) is

estimated based on available consequence information and judgment. The annual probability of flooding with non-breach life loss was estimated to be 1/500 AEP based on the top of levee (TOL) elevation (AEP_{TOL}), assuming the relative sea level change is the expected value between the high and low estimate in the year 2090. For low depths of overtopping, the non-breach life loss is still expected to be near zero. Overtopping would be an extreme event that is predicted in advance and the terrain and flood depths would likely allow for protective action to be successful.

1.2.3. Non-Breach Economic Risk

The annual exceedance probability (AEP) when flood damages begin to occur ($AEP_{s>0}$) is estimated based on available consequence information and judgment. The annual probability of flooding with non-breach economic cost was estimated to be 1/500 AEP based on the top of levee (TOL) elevation (AEP_{TOL}).

Non-breach damages for limited depths of overtopping are likely limited, similar to breach flows, due to the short duration of peak storm tides. However, for extreme events with multiple feet of overtopping depths, non-breach damages could reach hundreds of millions to billions of dollars of damages to structures and the contents of structures.

1.3. Major Findings and Understandings from the Risk Assessment

The following major findings and understandings were developed by the SQRA team:

- This risk assessment is limited to preliminary feasibility assumptions. Design submittal-specific abbreviated risk assessments will need to be performed at each 65% design submittal (or sooner if there is a specific need). A full system-wide risk assessment of the project will need to be completed at the end of construction.
- The current TSP is meant to be adaptive to sea level rise, but the decision to take a second action will be made in the future based on observed trends in sea level rise
- Availability of data is limited for this risk assessment, to include geotechnical data, wave observations, etc.
- Seismic modifications are driven by levee system performance (i.e., ground improvements under the alignment) and not to reduce structure seismic risks.
- Decreasing the length of flood risk management features and decreasing the complexity (e.g., number of transitions between different features) results in a reduction in risk.
- Drivers of high water at the project are primarily high tides combined with low pressure systems. Seasonal tides in January (i.e. "king tides") result in the highest tidal elevations. Low pressure systems can account for an additional rise of up to 3 feet. Rainfall runoff to the bay has some contribution to water levels, to a lower degree than pressure. Wind setup is more significant at

either end of the bay and is likely minimal at the project. Swell waves generated in the ocean generally dissipate to less than 0.5 feet as they propagate through the bay. Wind waves can be large, but are not typically associated with peak water levels at the project due to the weather patterns having opposing directions to critical wind fetches. Tsunami waves are expected to dissipate as they enter the bay and are not being considered for design of this project.

- After a major seismic event, if the project features are compromised, it may be warranted to order a mandatory evacuation of the population at risk of flooding. Conversely, after a major seismic event where bridges and tunnels may be compromised, it may be crucial that access to the wharves is maintained to provide supplies and transportation. These two needs may present opposing risks.

1.4. Recommendations

Tolerable Risk Guidelines (TRG) are used for risk-informed decision making within the USACE Dam and Levee Safety Programs. These guidelines are the criteria USACE uses to determine if the risks associated with dams and levee systems are “tolerable.” The four TRG are provided below.

- TRG 1 (Understanding the Risk): Society is willing to live with the risk associated with the dam or levee to secure the benefits provided by the dam or living and working downstream or in the leveed area.
- TRG 2 (Building Risk Awareness): There is a continuation of recognition and communication of the dam or levee risk.
- TRG 3 (Fulfilling Daily Responsibilities): The risks associated with the dam or levee system are being properly monitored and managed by those responsible for managing the risk.
- TRG 4 (Actions to Reduce Risk): There are no cost-effective, socially acceptable, or environmentally acceptable ways to reduce risks.

Recommendations were formulated based on these four TRG to manage and reduce risk. Additional details about the hazard, performance, and consequences can be found elsewhere in this report. The following recommendations were developed by the SQRA team.

13. Continue with the proposed plan to reduce flood risk through a comprehensive system of structural and non-structural measures. (All PFMs; TRG 1; PDT)
14. Develop comprehensive post-earthquake inspection plan for the CSRM. (All seismic PFMs; TRG 3; Sponsor)
15. Follow risk hierarchy for closure type (gates, fill placement, etc.) in the design of any closures, with up-and-over / crossing being first, swing gates and roller gates second, passive systems third, and sandbags last. (Closure PFMs; TRG 4; PDT)
16. Include plan to install, maintain, and update as needed instrumentation to track settlement and ground motions along the project alignment. These could include surveyed monuments, seismic monitoring, or other similar instrumentation. (All PFMs; TRG 3; PDT)

17. Develop project-specific ship impact design criteria to account for appropriate impact loads across the designed system. Consider the type of structure (levee, wall, etc.), the toe elevation, exposure to ship traffic, predominant winds and currents, etc. Consider dolphins, as needed, to reduce impact load requirements.
18. Update Emergency Action Plan (EAP) and create/add a Flood Warning Emergency Evacuation Plan (FWEPP) in the TSP, which can integrate into existing City/County emergency systems. Include breach and non-breach risk to underground parking structures and underground transit. Include specific forecasting and gaging to determine when transit is shutdown and when population is required to evacuate. Pier buildings may need to be evacuated if pier walls are loaded.
19. Consider the risk of additional expected consequences for underground occupants, damages, and loss of service (parking garages, BART, etc.)
20. Develop robust O&M plan that includes plan to inspect and maintain EWN coarse beach if using it to reduce structure heights based on effectiveness to reduce waves. Also include O&M of wave breaks as they are important to wave loading of northern reaches. If EWN options are pursued and provide flood risk reduction, they need to be monitored and maintained to the same degree as the FRM features. If EWN options are pursued only for environmental reasons, they can be allowed to adapt naturally, however, they should not be considered to provide a risk reduction. For example, an Ecotone levee option should not be used to reduce the levee height if the coarse beach has some likelihood of settlement or erosion.
21. Include two means of closure on all culvert outlets as a best practice to reduce backflow risks.
22. Include valves on both sides of levee for pressurized pipes that pass beneath levee or floodwalls.
23. Include monitoring plan to collect bathymetry along the toe of the alignment to identify any concerns of prop wash erosion or overdredging.
24. Include a temporary measure to "ring" the Embarcadero Station in the event the San Francisco Transition Structure fails and floods the BART tunnel. This measure could also be used in the interim to reduce risk of flooding over the existing bulkhead wall from entering the station entrance on the landside, prior to the end of construction.

2. Background

2.1. Location and Purpose

The low-lying assets and economic activity along the San Francisco Waterfront are at risk of flooding from coastal storms and extreme high tides, and from the potential failure of the century-old San Francisco seawall, which could result from structural deterioration or earthquake induced shoreline instability (liquefaction or lateral spreading). The San Francisco Bay Area is a seismically active region, and a major earthquake could happen within the study area at any time.

The risk of overtopping in the San Francisco Waterfront is exacerbated by rising sea levels. The rate of relative sea level change (RSLC) is uncertain and could rise from 1 to 10 feet by 2140 according to some of the predictive models. By 2040 under a High SLC curve, over 500 structures will be vulnerable to flooding from the 1% annual exceedance probability (AEP) event, which is an extreme storm with a 1% chance of happening in any given year. Some of these assets include San Francisco Municipal Transportation Agency (SFMTA) facilities and track, City facilities, and commercial real estate. Under the High SLC curve, average annual damages exceed \$100 million by 2046; this is driven both by potential damages from infrequent, high water level events, but also from repetitive flooding occurring in low-lying areas. By 2090 under the High SLC curve, there could be up to three feet of sea level change. This increases both the spatial extent of infrequent storm events and the effects of frequent flooding events.

A total of seven alternatives were developed as part of the San Francisco Waterfront Coastal Flood Study. These alternatives to reduce the coastal flood risk include different combinations of protections to the waterfront by considering what is necessary for four reaches of the project, such as no action or partial retreat. Figure 5 shows the study area and the four main reaches. The alternatives considered low, intermediate and high rate of RSLC scenarios. Some of the alternatives are adaptive and a subsequent action can be implemented in the future. After the alternatives were identified, analyses were performed to combine features of all alternatives into a plan that maximizes the benefits. The Tentatively Selected Plan (TSP) is a cost effective, hybridized plan that combines retreat and defend measures, scaled to perform under the lowest initial risk and to adapt to risk of a higher rate of RSLC as a potential subsequent action.

While there are plans for second action in 2090 that depends on different RSLC scenarios, this risk assessment report focuses on the first or initial action of the 2040 TSP. Another risk assessment would need to be performed in the future for subsequent actions.

San Francisco Waterfront Storm Damage Risk Reduction System



Figure 3. San Francisco Waterfront Coastal Flood Study Area.



Figure 4. Reaches of the study area.

2.2. Project Description and Pertinent Data

The main components of the project are shown in Figure 7 and include 4 reaches: Fisherman's Wharf, Embarcadero, South Beach/Mission Bay and Islais Creek Bayview. A broad description of the initial actions of the TSP is shown for each one of the reaches. Drawings depicting the key features of the project are included in Appendix B.



Figure 5. Tentatively Selected Plan – Initial Actions.

Table 1 shows the array of alternative plans that were developed for the San Francisco Waterfront. Combinations of these alternatives were used to develop the TSP. Table 2 illustrates conceptual framework for the initial or first action (2040) and the range of second actions (2090). This range considers low, intermediate and high rate of RSLC scenarios. As mentioned before, the focus of this report is on the first action. Physical quantities and extents of the TSP features are summarized in Table 3.

Array of alternatives for San Francisco Waterfront.

Table 1.

Array of alternatives for San Francisco Waterfront.

Alternative	Description
Alternative A	No Action
Alternative B	Nonstructural (e.g., floodproofing)
Alternatives C, D	Alternative C: Defend, Scaled for Lower Risk (low rate of SLC) Alternative D: Defend, Scaled for Low-Moderate Risk (intermediate rate of SLC)

Alternative	Description
Alternatives E, F, G	Alternative E: Defend Existing Shoreline, Scaled for Higher Risk (high SLC) Alternative F: Manage the Water, Scaled for Higher Risk (high SLC) Alternative G: Partial Retreat, Scaled for Higher Risk (high SLC)

Table 2.
TSP first and second actions.

Reach	First Action (2040)	Second Action Low RSLC (2090)	Second Action Intermediate RSLC	Second Action High RSLC
Fisherman's Wharf, Reach 1	Alternative B	No action	Alternative B (Additional NS)	Alternative G 19' NAVD88
Embarcadero, Reach 2	Alternative G 15.5' NAVD88	No action	No action	Alternative G 19' NAVD88
South Beach/Mission Bay, Reach 3	Alternative D 13.5' NAVD88	No action	Alternative D 15.5' NAVD88	Alternative E 19' NAVD88
Islais Creek/Bayview, Reach 4	Alternative D 13.5' NAVD88	No action	Alternative D 15.5' NAVD88	Alternative E 19' NAVD88

Table 3.
Physical quantities and extents of 2040 TSP first actions.

Feature	Volume (CY)	Area (SF)	Length (Miles)
Levees	98,979	-	2.92
Combi Wall	33,788	-	1.96
T-Wall	979	-	0.78
Wharf	-	609,840	-
Building Raise*	-	326,435	-

*Non-structural measures not assessed as part of the risk assessment

Coastal flood hazards relevant to the 7.5-mile San Francisco Waterfront study area are organized below into four interrelated categories: coastal flooding, inundation, waves, and erosion.

- Coastal flooding occurs when Bay water levels rise above the shoreline along the waterfront, overtopping the shoreline and temporarily flooding inland areas. Most of the developed areas along the shoreline are built on Bay fill, are generally low-lying and flat, and only a few inches to a few feet above the Bay's existing highest annual tides. Because of this, the extent of flooding and the potential damage and disruption that can occur are sensitive to small changes in Bay water level elevation (e.g., ± 6 inches) once the shoreline is overtopped. Coastal flooding already occurs approximately annually along the lowest spots of the shoreline, such as near Pier 14 by the

Agriculture Building, where Bay water levels and waves overtop the shoreline and cause disruption of pedestrian and vehicle traffic along the Embarcadero promenade and roadway.

The existing risk of coastal storm damages are currently isolated to low-lying areas along the waterfront where land elevations are less than +10 feet North American Vertical Datum of 1988 (NAVD88), which is the average height of the seawall. The height of the seawall varies, as do heights along the entire waterfront. Specific low points along each reach are shown in Table 4. An example of past overtopping of the seawall occurred during an extreme tide in November 2015, resulting in Bay waters flooding the Embarcadero promenade along the waterfront near the Ferry Building. This event damaged steel plate joints, shut down at least one lane of traffic along the Embarcadero corridor, and posed a safety hazard to pedestrians. Additionally, in March 2023, a winter storm with high winds, led to extreme wave activity causing substantial shoreline overtopping at low tide (Figure 8).



Figure 6. Wave overtopping for existing conditions, March 2023.

Table 4.
Elevations of existing low points.

Feature	Elevation (ft-NAVD88)
Reach 1	9.7 ft
Reach 2	8.1 ft
Reach 3	6.6 ft
Reach 4	6.9 ft

- Coastal inundation refers to the permanent inundation of land by high tides, such as areas that are below mean higher high water. Under existing conditions, only the wetland areas, including fringe wetland areas along Islais Creek and Mission Creek are inundated on a regular basis. However, as sea levels rise, the area inundated by the high tides will increase.
- Waves in the Bay include both longer period ocean-driven swell propagating through the Golden Gate, and locally generated, wind-driven waves. Ocean-driven swell waves propagate parallel to the northern San Francisco shoreline and may pose a hazard to the Aquatic Park municipal pier and the port's finger piers. Although these waves temporarily increase Bay water levels, they are not a significant direct wave hazard along the shoreline.
- Coastal erosion occurs when currents and waves wear down and carry away earthen or engineered materials along the shoreline. In areas with a natural or naturalized shoreline, like the Pier 94 Wetlands or Heron's Head Park, erosion can lead to inland migration of the shoreline. For engineered coastal defenses (e.g., a floodwall or seawall), erosion is of particular concern at the toe of the structure, because erosion can weaken the foundation of the structure and increase the risk of failure.

Four broad, conceptual, high-level approaches – Defend, Accommodate, Retreat, and Hybrid – were developed as a basis for formulating plans to address coastal flood risk. By law, at least one entirely nonstructural plan is required. The Defend, Accommodate, and Managed Retreat approaches are illustrated in Figure 9. Each reach has at least one of these strategies.



Figure 7. Approaches to reducing risk.

2.3. Reach 1

The TSP for Reach 1 (Fisherman's Wharf) includes "floodproofing" buildings (Figure 10). There is risk to the occupants of the buildings based on the effectiveness of the final floodproofing plan, however,

there is no effect on the risk to the public based on the performance of these non-structural features. Therefore, these non-structural measures were not assessed in the risk assessment.

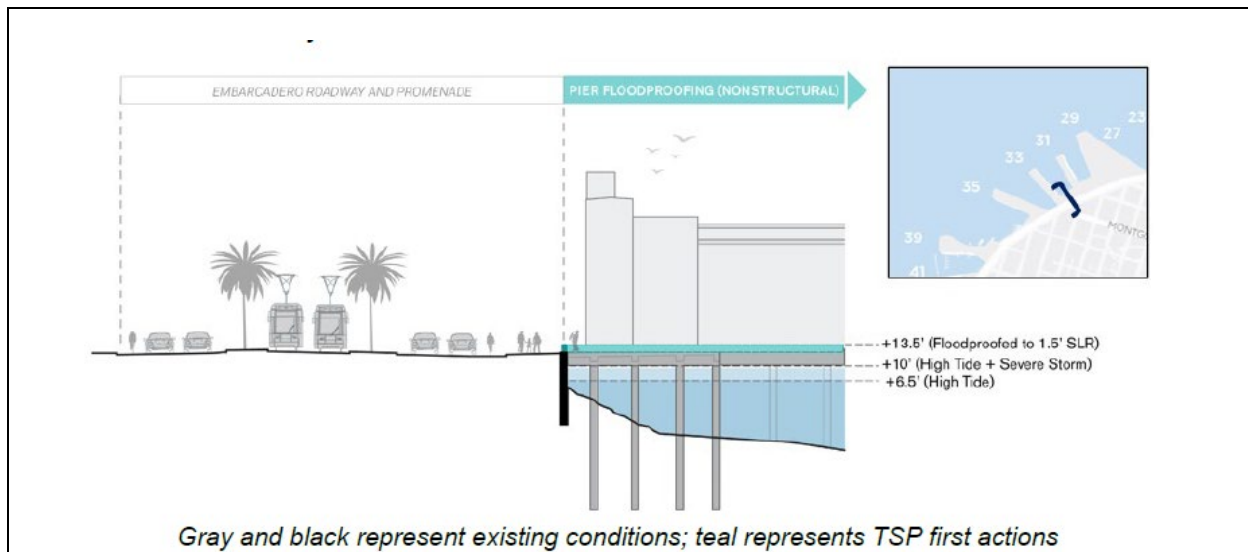


Figure 8. Reach 1 proposed non-structural measures.

2.4. Reach 2

Reach 2 (Embarcadero) includes ground improvements and modifications to the wharf wall/ bulkhead wall. Ground improvement will be performed to mitigate lateral spreading and liquefaction hazards along the coastal flood defense alignment to meet the required seismic performance objectives. In addition, the shoreline along the Embarcadero will be raised by 3.5 to 7.5 feet to defend against 3.5 feet of SLR (finish elevation of 15.5 feet NAVD88). Some of the buildings in wharves will be elevated as well. Figure 11 shows examples of the existing conditions and the proposed improvements along Reach 2.

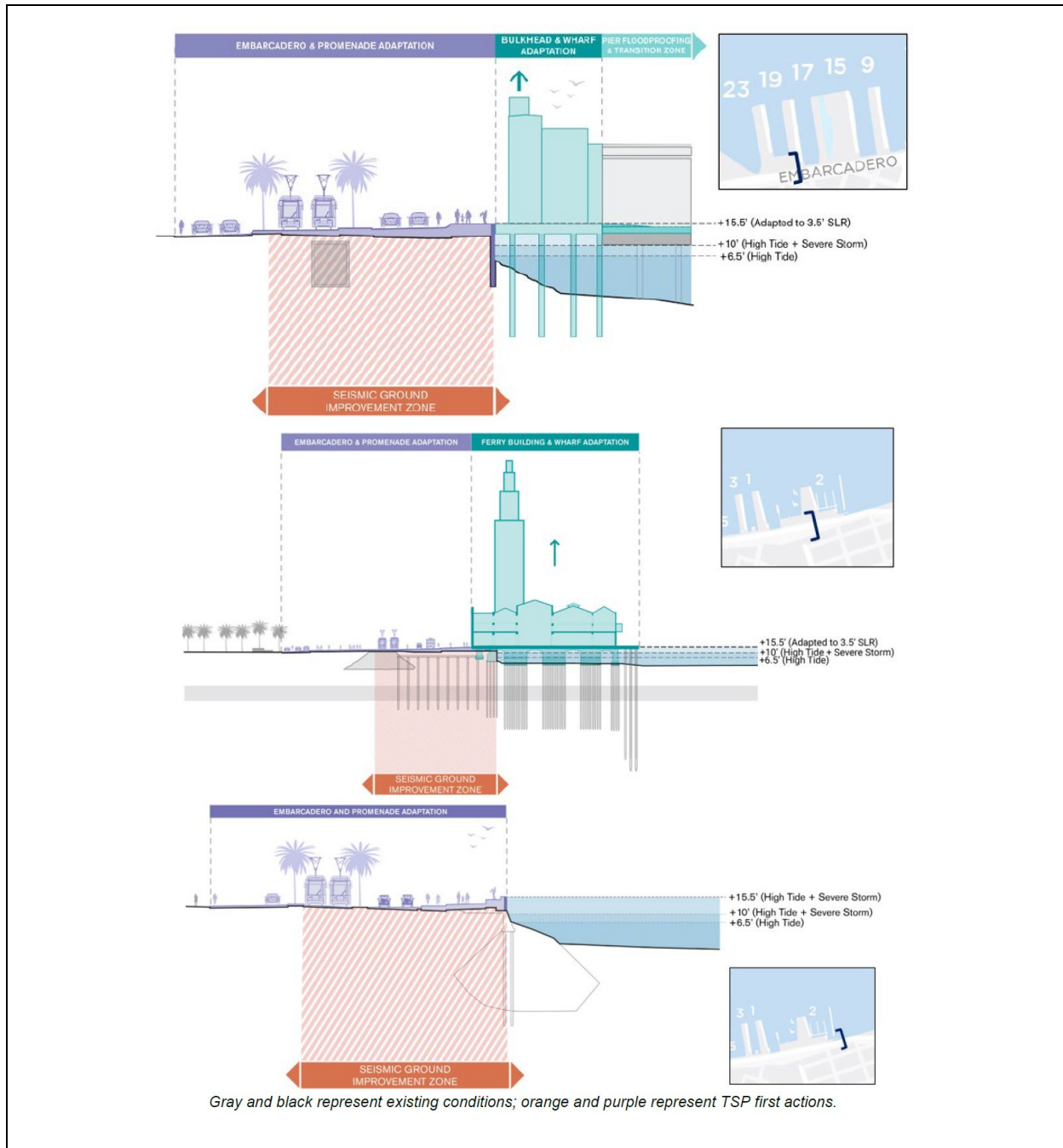


Figure 9. Reach 2 proposed features.

2.5. Reach 3

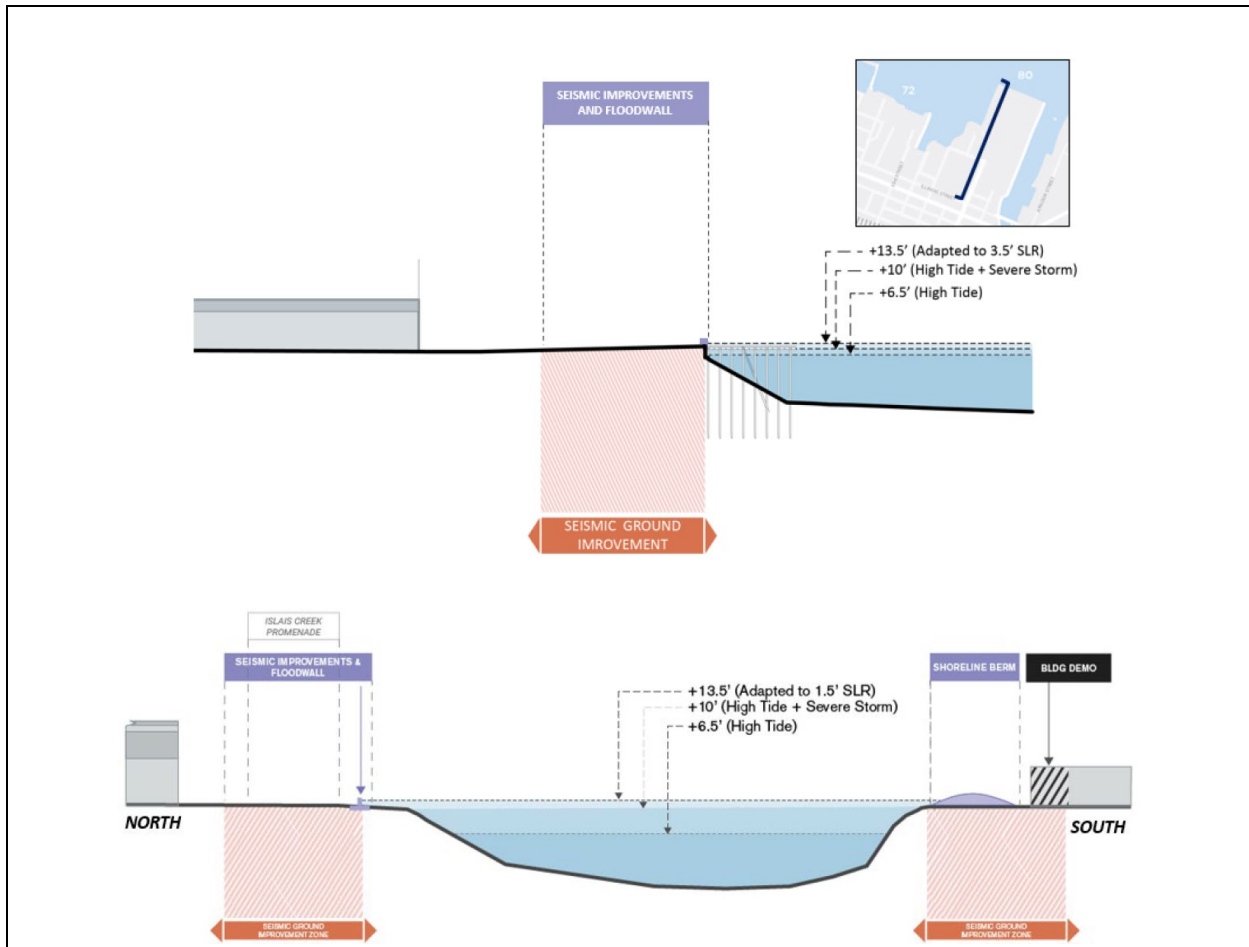
Reach 3 (Mission Creek/Mission Bay/South Beach) includes ground modifications, modifications to the wharf wall/bulkhead wall, and levees. Similar to Reach 2, ground improvement will be performed to

Gray and black represent existing conditions; orange, pink, and green represent TSP first actions

Figure 10. Reach 3 proposed features.

2.6. Reach 4

Lastly, Reach 4 (Islais Creek/Bayview) will also include the ground improvement and will elevate the shoreline to defend against 1.5 feet of SLR (13.5 feet NAVD88). This reach will also include floodwalls, curb extensions and levees that will tie into high ground. This reach will have the most NNBFs of the entire project. Figure 13 shows the examples of the first actions at Reach 4.



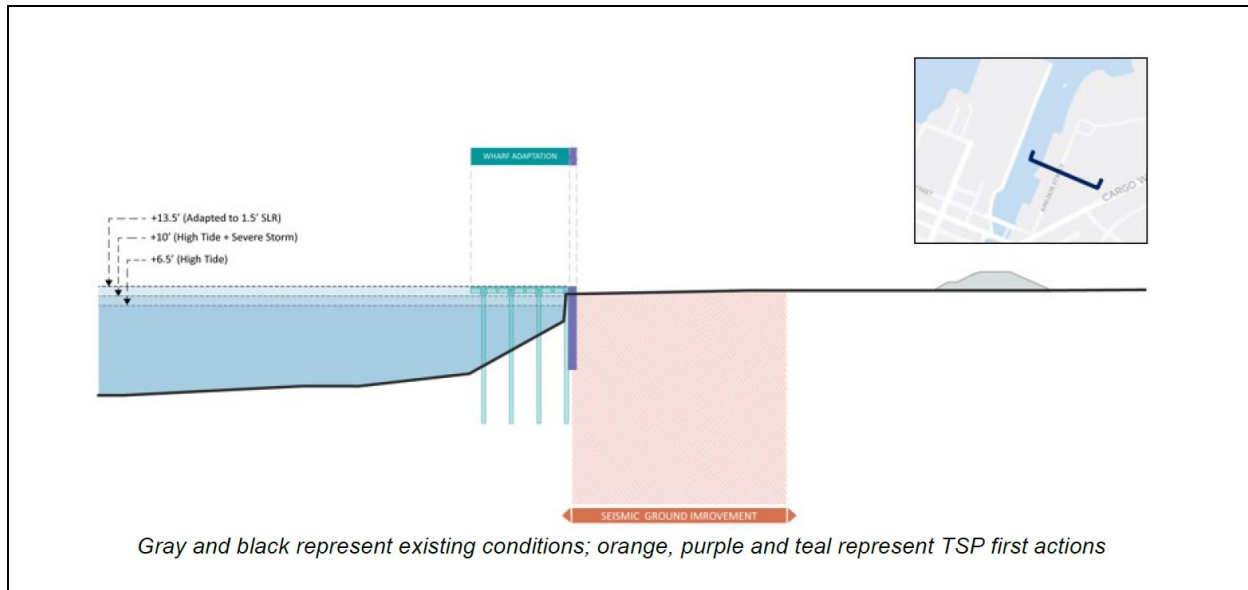


Figure 11. Reach 4 proposed features.

2.7. Features

The following features were assessed as part of the risk assessment.

2.7.1. Levees

Levees (Figure 12) would be constructed of imported impervious earthen fill material, assumed to be procured from a commercial source near San Francisco. The side slopes of the levees would have side slopes no steeper than 3H:1V, but isolated areas will include side slopes ranging from 5H-10H:1V for vehicle pass. The required levee height and width is dependent on local existing elevations, the final flood side design side slope and configuration, and the crown width assumed to vary between 10 to 20 feet. In addition to regular levees, ecotone levees will be constructed with the flood side slopes gently sloped towards a tidal marsh. They will connect the levee to the marsh surface and can provide high quality transition zone habitat when vegetated with appropriate native plants.

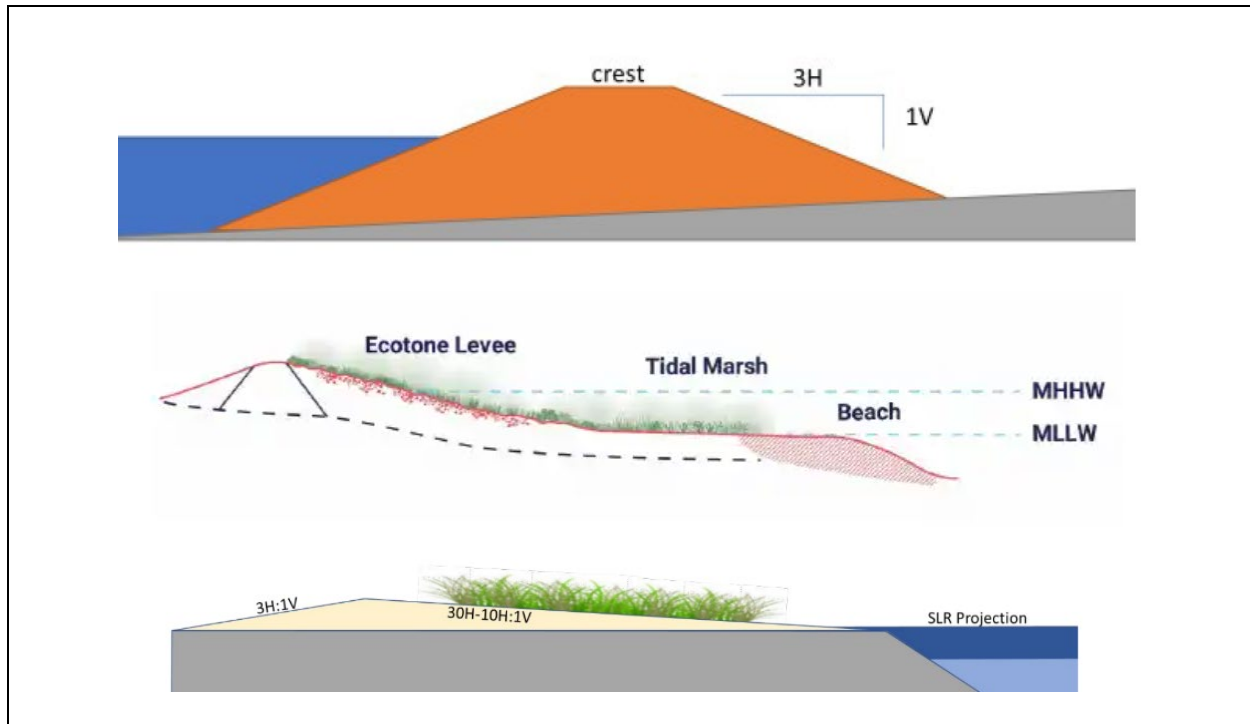


Figure 12. Typical levee sections and ecotone levees.

2.7.2. Floodwalls (T-walls, L-walls)

Where the chosen flood protection measure needed to be a structural wall due to real estate constraints and the required flood protection height is about 4 feet or greater, either a concrete T-wall or L-wall will be utilized for the protection measure. At design level, the T-walls and L-walls are assumed to have sheet pile cutoffs and to be founded on deep foundations consisting of either steel H-piles or pipe piles which could be driven either vertically or battered depending on the design loads and underground site constraints.

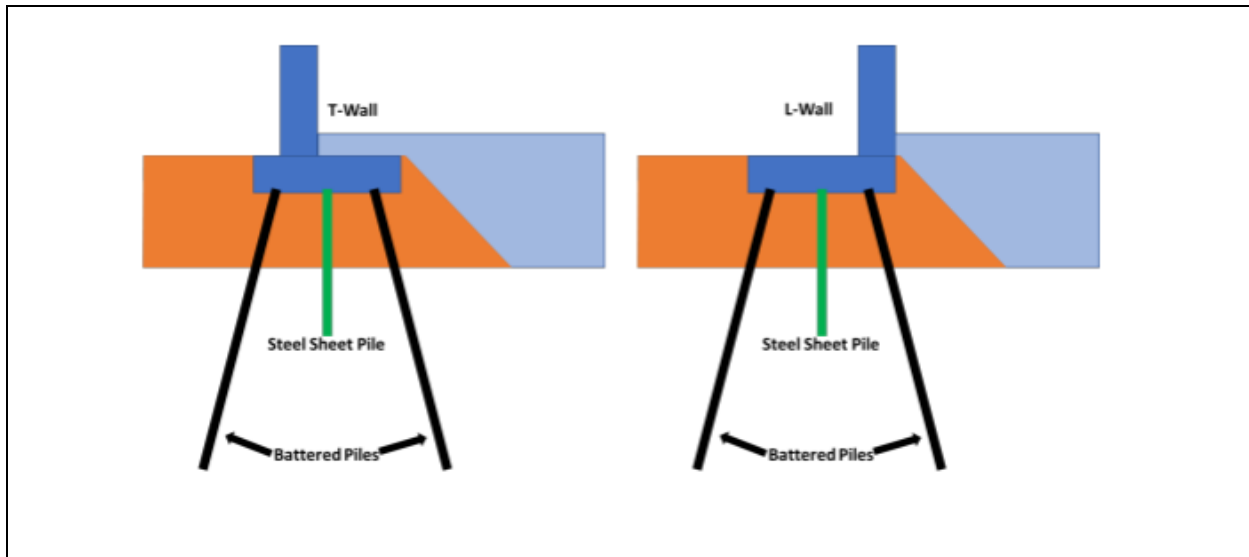


Figure 13. Examples of T-walls and L-walls.

2.7.3. Wharves (*wharf wall, pier wall, curb walls, road raises*)

The Embarcadero seawall was built over 100 years ago. The seawall comprises 22 unique sections, some with or without a bulkhead wharf, and approximately 22 remaining finger piers. Much of the seawall was built after 1900, sometimes in conjunction with a bulkhead wharf structure, typically adding 40 feet of additional space over the water. The first actions of the TSP contemplate elevating the wharves and raise the roads as well. Wharf walls and pier walls will be constructed as part of the project. There are lengths of the reaches that comprise existing bulkhead piers or wharves which are currently at elevations of approximately 11 to 12 feet. The most cost-effective way to raise these structures by up to about 2.5 feet for flood protection is a curb wall. Figure 14 shows examples of the wharf walls (seawall) and curb walls.

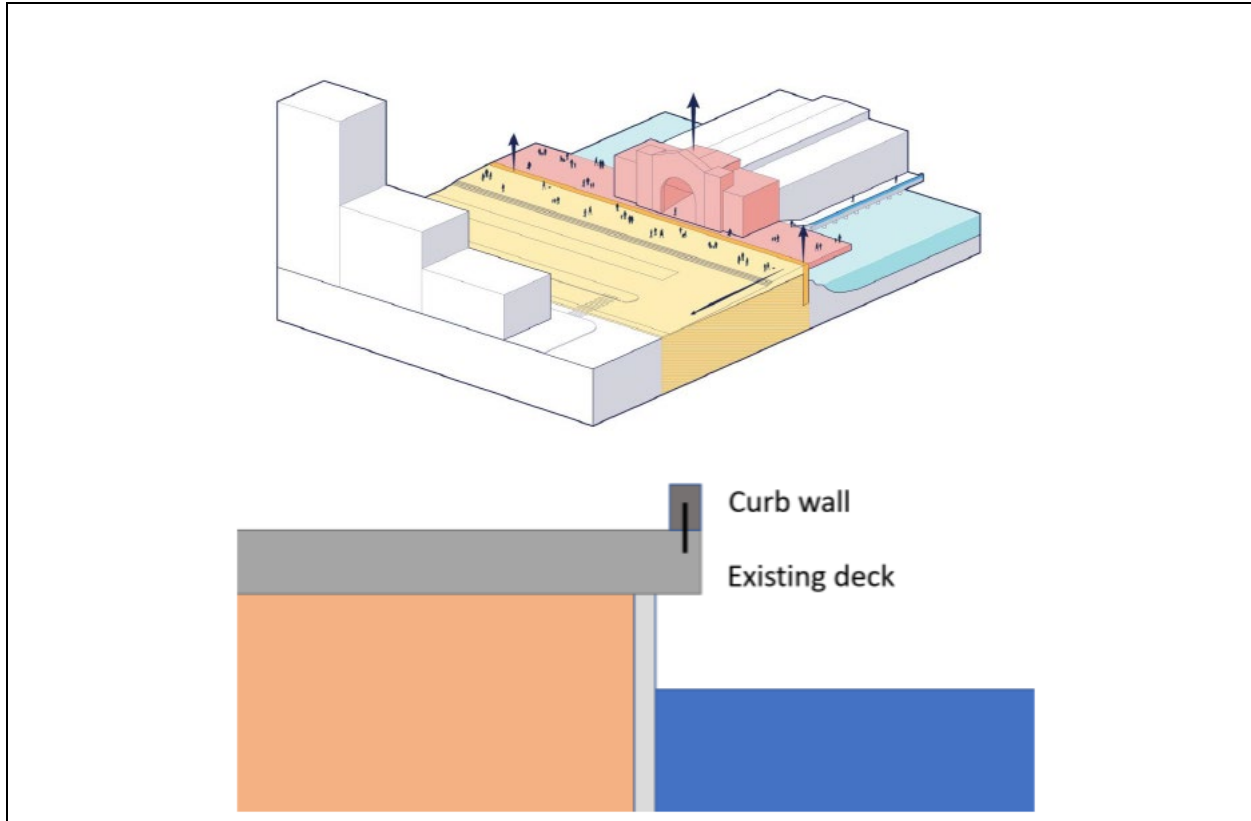


Figure 14. Wharves and curb walls.

2.7.4. Closures

Closures evaluated for the risk assessment include all gates that are required to complete the line of flood protection. The design has not progressed enough to determine how many closures and what type of gates will be in place. Swing gates and roller gates have been considered where flood protection is required on an existing roadway, bridges, train tracks, or where access is required to the piers. Larger storm surge gates such as sector gates or miter gates would be required in other areas to restrict water surge from reaching large areas of the shorelines. Figure 15 shows examples of closures considered for the project.

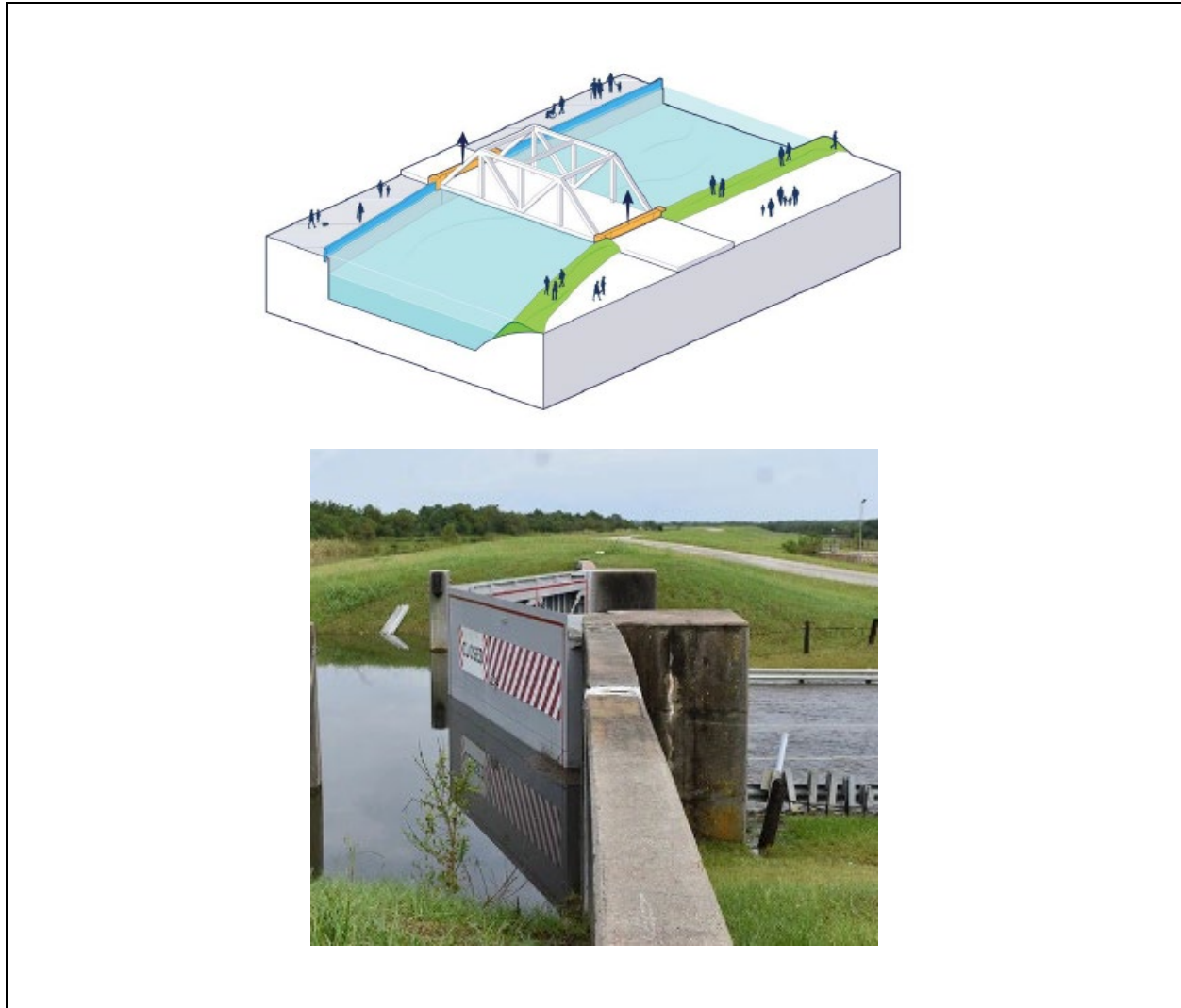
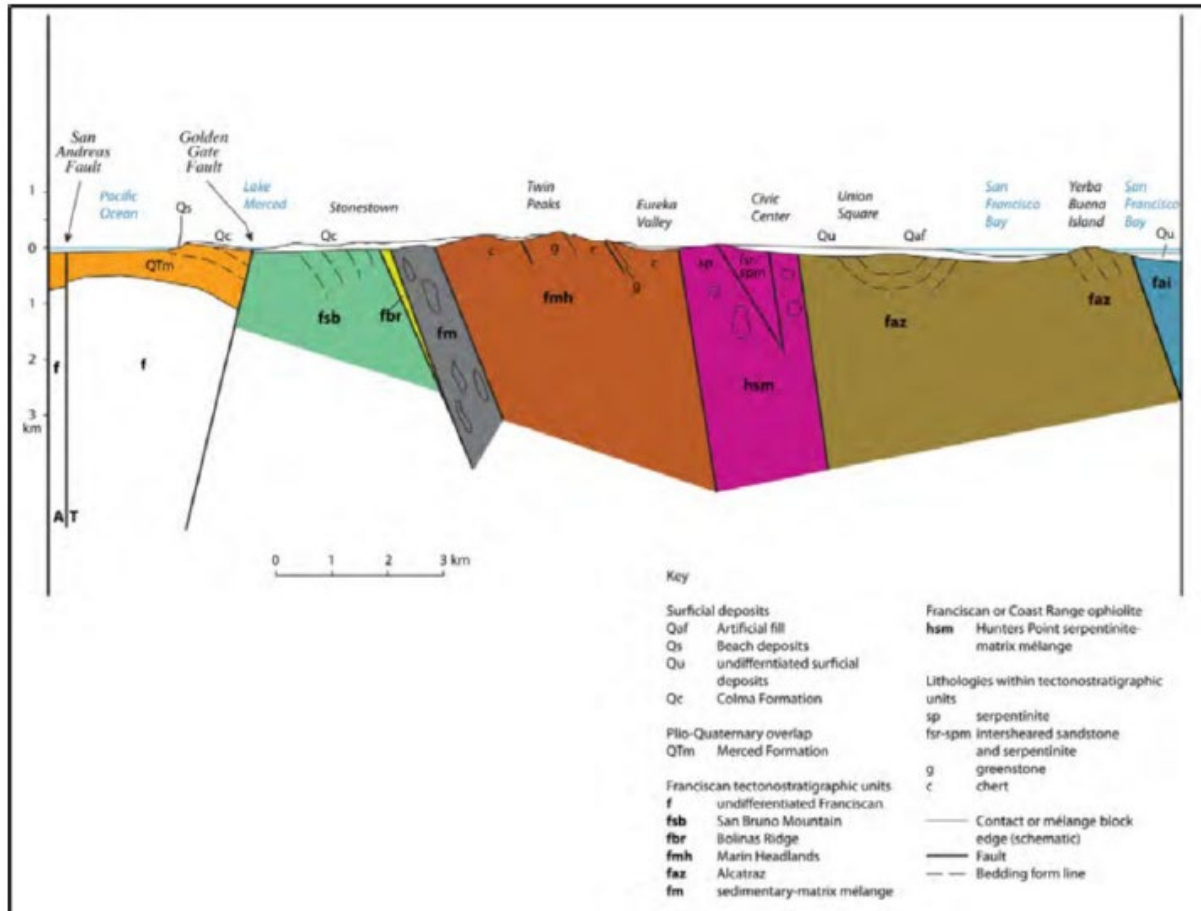


Figure 15. Examples of closures.

2.8. Site Characterization

The project site is located at the north-eastern tip of the San Francisco Peninsula, a 7-mile wide point of land that divides the Pacific Ocean from southern San Francisco Bay. The geologic formations in the San Francisco Bay range in age from Jurassic to Recent Holocene. The Franciscan Complex is the oldest and underlies younger surficial deposits throughout the San Francisco Bay region. The Franciscan Complex consists mainly of marine-deposited sedimentary and volcanic rocks. Uplifted areas in the valleys formed in the Bay Area eroded which resulted in Pleistocene and estuarine sediments being deposited. The city of San Francisco is underlain by Mesozoic rocks of the Franciscan Assemblage. In north-eastern San Francisco, the bedrock belongs to the Alcatraz Terrane and they form the bedrock foundation beneath the Quaternary deposits along the shoreline (Figure XX).



Source: Bartow and Johnson 2018

Figure 16. Geologic cross section of San Francisco from Yerba Buena Island to the Pacific Ocean through Twin Peaks and Lake Merced

Mesozoic bedrock directly underlies the hills and peaks and is covered by Quaternary-age alluvium, nearshore deposits, sand dunes, and artificial fill in the flatlands. Similarly, in the offshore, the bedrock is overlain by Quaternary deposits. Offshore deposits in the project area consist primarily of sequences of estuarine muds, local alluvium, and sandy-silty sediment brought into the bay from the Sacramento River. The strong influence of Quaternary glacial cycles explains the stratigraphic units beneath the existing seawall.

2.8.1. Geologic Units

Previous studies of the sediments underlying the project area shows the presence of alternating coarse-grained and fine-grained sediments and have linked them to major glacial cycles. The estuarine deposits in the Bay area are intercalated with alluvium and dune sand. The stratigraphy at the project site has a correlation to glacial-interglacial stages. The key native geologic units are Franciscan Complex Bedrock, Lower Layered Sediments, Old Bay Mud, Upper Layered Sediments and Young Bay Mud. Lastly, Artificial Fill is on top of the Young Bay Mud.

Table 5. Major geologic units in San Francisco.

Stage or Time Period	Symbol	Unit Name	Description
Historical	Af	Artificial fill, rock dike	Heterogeneous sandy fill, and rock dike of angular sandstone rubble
Holocene (interglacial)	YBM*	Young Bay Mud	Very soft to firm estuarine clay and silt with minor sand layers
Wisconsin (glacial)	ULS*	Upper Layered Sediments	Layered sequence of sand, clays, and occasional gravels of alluvial and eolian origin, capped by beach sands
Sangamon (interglacial)	OBM	Old Bay Mud	Dense estuarine clay and silt with interbeds of sand
Illinoian (glacial and earlier Quaternary)	LLS	Lower Layered Sediments	Layered sequence of sand, gravel, silt, and clay of alluvial, estuarine, and colluvial origin
Mesozoic	KJf	Franciscan complex	Sandstone and shale bedrock, weathered

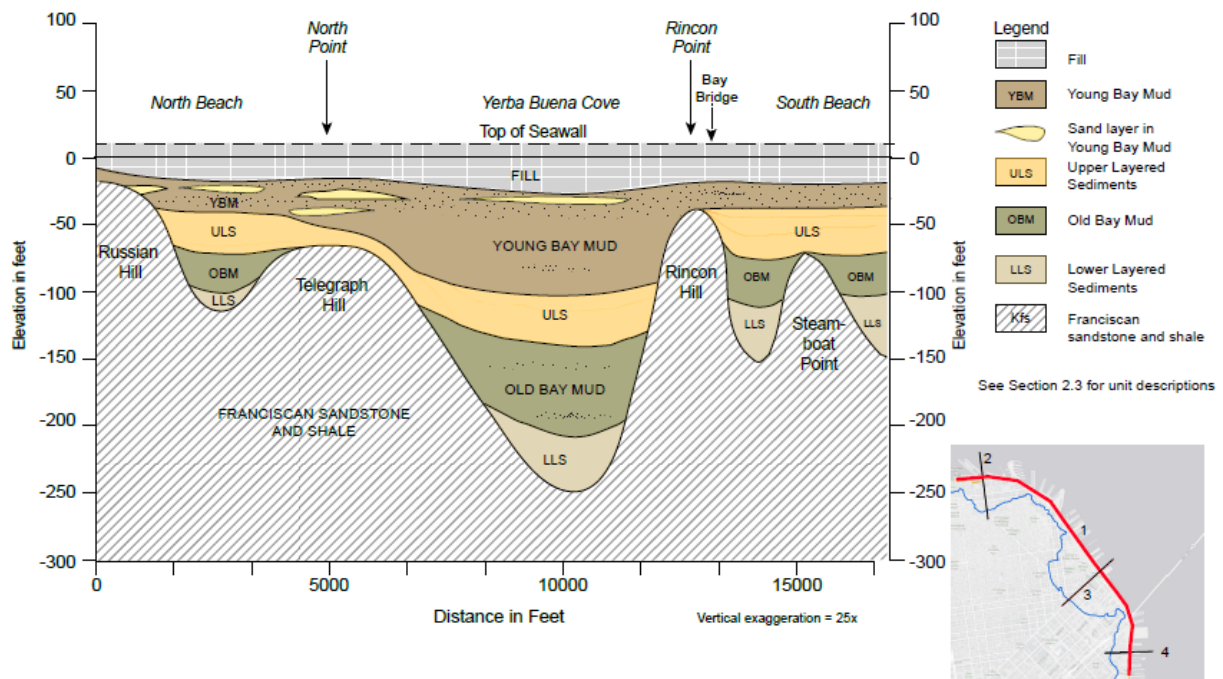


Figure 17. Conceptual geologic model along existing seawall alignment.

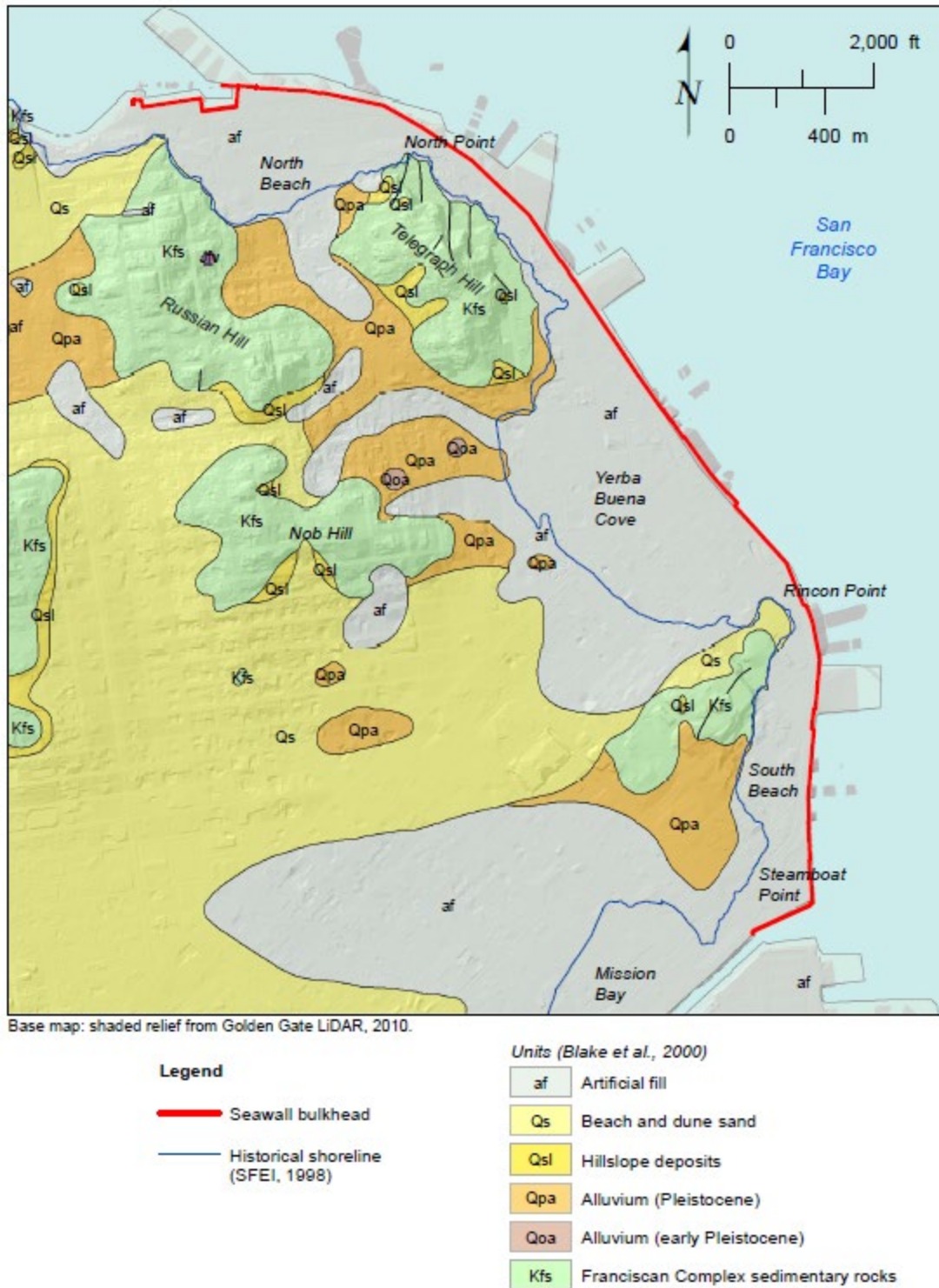


Figure 18. Geologic Map of North-Eastern San Francisco.

2.8.1.1. Lower Layered Sediments

The Lower Layered Sediments include soils of estuarine, alluvial, and colluvial deposition which were deposited approximately 120,000 to 400,000 years ago during older glacial–interglacial cycles. These soils include poorly graded gravel (GP), clayey gravel (GC), poorly graded sand (SP), silty sand (SM), clayey sand (SC), fat clay (CH), and lean to sandy clay (CL). The fine-grained soils are generally stiff to hard, and the coarse grained soils are typically dense to very dense based on in-situ test results. Total densities range from approximately 115 to 140 pounds per cubic feet (pcf).

2.8.1.2. Old Bay Mud (or Yerba Buena Mud)

The Old Bay Mud was deposited approximately 75,000 to 120,000 years ago during an earlier sea-level high stand that was the most recent predecessor to today's San Francisco Bay. This deposit consists of a sequence as much as 100 feet thick of relatively homogenous gray, marine silty clay, and occasional thin laterally discontinuous lenses of fine sand and shell-rich horizons. This deposit is generally present across the project site but is absent in some areas with shallower bedrock. Per Bartow and Johnson 2018, this deposit generally consists of interbedded stiff to very stiff clay (CH and CL) [N-values typically in the range of 10 to 60] and dense to very dense sand (SP) and silty sand (SM) [N-values typically in the range of 30-100+]. Total densities range from 100 to 135 pcf.

2.8.1.3. Upper Layered Sediments

The Upper Layered Sediment was deposited approximately 8,000 to 75,000 years ago as intermediate sea levels existed associated with entering and exiting the most recent glacial period. This deposit lies just above the Old Bay Mud. The deposits are dominantly of alluvial and eolian origin and are typically capped by a beach sand representing the encroachment of the shoreline in the early Holocene. Per Bartow 2018, the soils in this deposit are generally classified as poorly graded sand (SP), fat clay (CH), or lean to sandy clay (CL). The fine-grained soils are generally medium stiff to very stiff, and the coarse-grained soils are typically medium dense to very dense based on in-situ test results. Total densities range from 110 to 139 pcf.

2.8.1.4. Young Bay Mud

The poorly consolidated Holocene (younger than about 8,000 years) marine deposits commonly known as Young Bay Mud (YBM) were deposited during the most recent sea level intrusion coincident with present-day San Francisco Bay. The deposit occurs as a surficial blanket of fine sediments over the floor of the San Francisco Bay and the quiet water coves along the bay shoreline. The deposit is generally less than 165 feet thick. Within the San Francisco city limits, YBM is located along the margins of the Bay, between the modern shoreline and historical limit of the tidal marsh, and generally buried by AF. During construction of the Embarcadero seawall in the late 1800s and early 1900s, a wedge along the wall alignment of this deposit was dredged to approximately elevation -35 feet and backfilled with rock or sand upon which the seawall was constructed.

YBM is typically a clay that is highly compressible. It also can contain interbeds of alluvial fine sand lenses originating from nearshore streams and creeks along the margins of the current and historical bay shoreline. Per Bartow and Johnson 2018, the fine-grained soils are typically classified as lean clay (CL), fat clay (CH), or elastic silt (MH) with varying amounts of sand, shells, and organics. The coarse-grained soils are typically classified as silty sand (SM), poorly graded sand (SP), sandy or clayey silt (ML), and clayey sand (SC). The fine-grained soils are generally very soft to medium stiff, and the coarse-grained soils are typically loose to medium dense based on in-situ test results. Fine-grained clay strata within the

deposit generally have Atterberg limits values with liquid limits ranging from 18 to 89 and plasticity index range from 0 to 58. The in-situ water contents in this unit are typically closer to the liquid limit than the plastic limit of the soils, consistent with the clayey soils being relatively soft and normally consolidated. Total densities range from <90 to 130 pcf.

2.8.1.5. Artificial Fill

Since the 1800s the northeastern and eastern margins of the city of San Francisco have undergone significant landscape modification through multiple episodes of fill placement. Areas receiving the most extensive AF include inlets, coves, and saltwater marshes. Filling of the bay margins included a variety of fill placement methods ranging from dumping to hydraulic filling, as well as the intentional sinking of abandoned wooden sailing vessels. AF in the city includes a combination of local native sediments, including hydraulically placed dune sand and bay sediments, and miscellaneous construction debris, including brick, concrete rubble, metal, glass, and timber all of which were typically placed on top of weak YBM. In Yerba Buena Cove, AF includes debris from Gold Rush-era ships that were sunk and used as fill. Following the 1906 earthquake and fire, significant amounts of demolished structural debris was disposed as fill along the bay margin north and east of Market Street. Because of the different source materials and methods used for placement, the AF is highly variable and depends heavily on the source of the fill material. For instance, material sourced from the sand dunes and dredged bay sediments typically consists of loose, poorly graded sand to clayey sand and soft clay and often may be mixed or overlie rubble such as brick, asphalt, concrete, wood, broken rock, and scattered gravel.

The fill materials consisting of soils are generally classified as poorly graded sand (SP), silty sand (SM), or sandy clay (CL). The fine-grained soils are generally soft to medium stiff, and the coarse-grained soils are typically very loose to medium dense based on in-situ test results. These soils are generally normally consolidated and due to the high variability of the composition of the deposit matrix are subject to high variability in settlement potential within short distances.

3. Periodic Inspection

The semi-quantitative risk assessment did not include a periodic inspection. Therefore, this chapter is left blank intentionally.

4. Hydrologic Hazards

4.1. Introduction

USACE uses a risk-informed approach where risk assessments and deterministic design standards provide more information for decision makers. The primary objectives of the hydrologic hazard analysis to support a semi-quantitative risk assessment (SQRA) are:

- Characterize the dominant flood hazard(s) for the watershed and system and document/describe the history of past flood events.
- Characterize the typical duration of flood events and the annualized water surface elevation-duration relationship.
- Provide water surface elevation-frequency relationship at numerous index locations throughout the system and through development of system water surface profiles along the line of defense.
- Document other known hydrologic and hydraulic concerns and uncertainties for the line of defense.

4.2. Background Data

The current waterfront consists of multiple shoreline structures. These consist of the following:

- Fishermans Wharf
- Embarcadero Seawall – Built over 100 years ago and consists of 22 unique sections and approximately 22 remaining finger piers that span approximately 3 miles of waterfront. The marine structures were constructed with a variety of materials and construction styles based on when completed. This area includes the Ferry building and plaza.
- Large Concrete Pile Structures
- Timber Piled Shoreline Wharves
- Timber Piles with Concrete Jackets
- Square Precast Piles and Concrete Deck

For more detail on these structures and their locations along the waterfront see Appendix B in the Draft Feasibility report.

All data included in this study was done in NAVD 88.

The San Francisco Waterfront Flood Study project area consists of the bayside waterfront from approximately Fisherman's Wharf to Herons Head. The study area reach delineations and near by gage locations are shown in Figure 16.

The study area is divided up into four primary reaches, shown in Figure 1. Reach 1 defines the northern waterfront and consists of Aquatic Park, Fisherman's Wharf, Pier 31 to Pier 35, and the North Beach neighborhood. Reach 2 consists of the Embarcadero which spans from international cruise terminal to Pier 27 to the San Francisco-Oakland Bay Bridge. Reach 3 consists of the Mission Bay and Mission Creek area and spans from the San Francisco-Oakland Bay Bridge and the former Potrero Power Plant. Reach 4 consists of Islais Creek and Bayview and spans from Pier 80 in the Potrero Hill neighborhood near the former Potrero Power Plant to Herons Head.

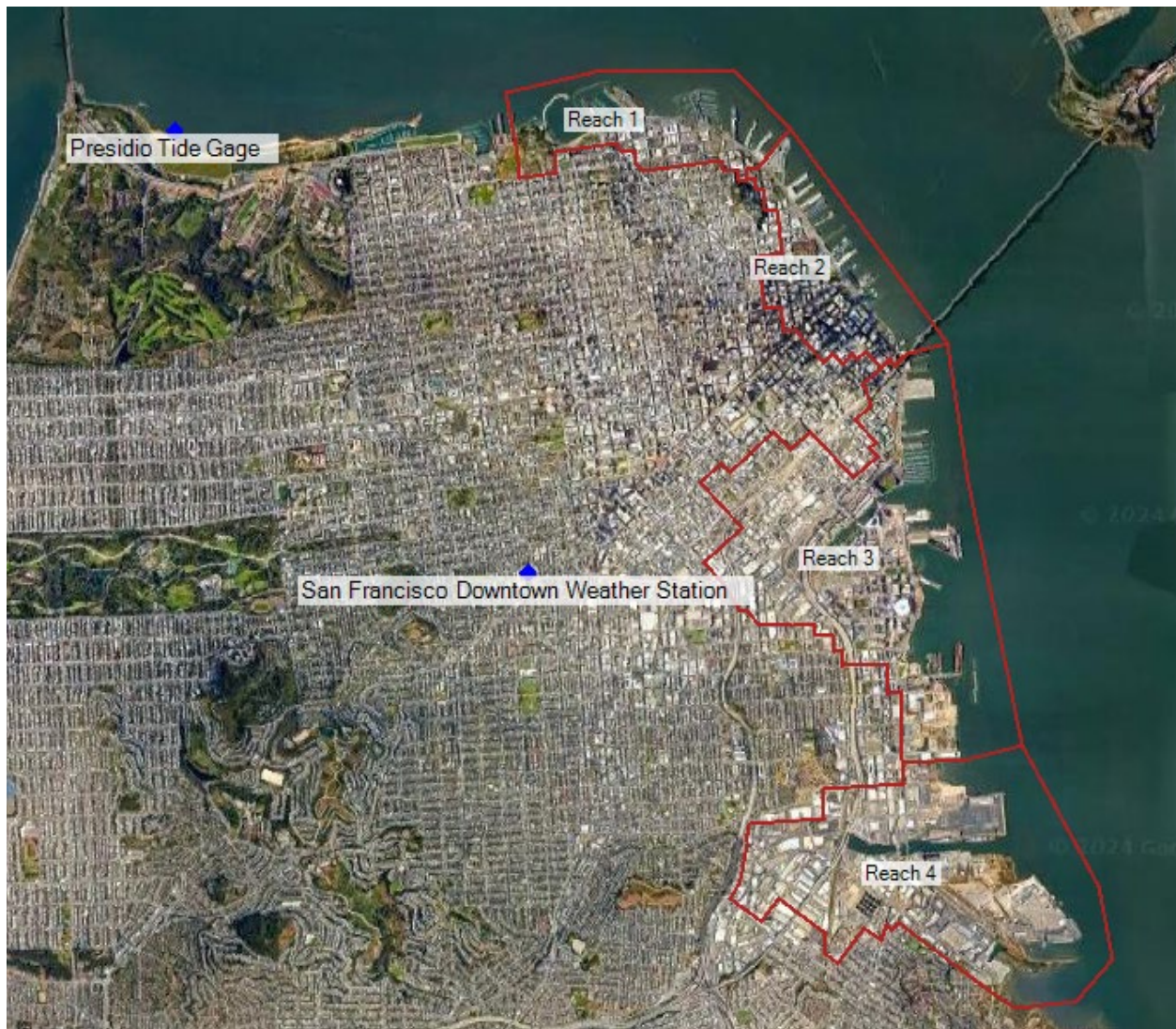


Figure 19. Overview of Model Reaches and Location of NOAA Presidio Gage and San Francisco Downtown Weather Station

4.2.1. Data Sources

The Presidio Tide Gage is Station 9414290. Located near the Golden Gate Bridge, the Presidio Tide Gage is the oldest continually operating tidal gage in the Western Hemisphere with the first bit of data being recorded on June 30, 1854.

Precipitation data is available from the 1908 through 2018 water years. Hourly data is gathered from the Downtown San Francisco National Oceanic and Atmospheric Administration (NOAA) gage (COOP:047772; 1908-2011) and the San Francisco SFPUC Mission Street gage (ID: RG31; 2011 – 2018) to get a 110-year period of record for the downtown area. The Mission Street gage is a combination of NOAA data through December 2011 and SFPUC gage data from January 2011 through 2018. The SFPUC records data in tips and millimeters which was converted to inches using the SFPUC methods.

FEMA relied on a regional MIKE21 Flow (HD) and Spectral Wave (SW) hydrodynamic and wave dynamic numerical model of the Bay to develop a 31-year continuous timeseries of water levels and waves, 1973 through 2004.

4.2.2. Tide Water Levels

The water levels of the Bay are highly dynamic, with an average daily tide range of approximately 6 feet near the Golden Gate. Due to the complex bathymetry and geographical configuration of the Bay, the tide range and elevation vary spatially. Interactions among tidal processes (for example, reflection of tidal waves), bathymetric variations, and shoreline orientation amplify the tides with increasing distance from the Golden Gate inlet. The tide range also varies along the same gradient, with a 6-foot tide range near the Presidio tide gage, shown in Figure 17, and an 8 foot tide range, in the far south bay. Even on a smaller scale along the San Francisco shoreline, the elevation of MHHW varies by approximately 0.5 foot between Aquatic Park and Heron's Head Park along the 7.5-mile study area shoreline.

Except for King Tides, which are predictable astronomical tides, extreme water levels represent a temporary, short-term (hours to months) increase in sea level above the predicted astronomical tide level. This difference in water elevation between the predicted and observed tides may include storm surge, El Niño and/or Pacific Decadal Oscillation cycles, local wind setup, freshwater inflows, or a combination of these factors. Observations of extreme water level at tide stations typically do not include short-term wave effects, although wave effects can also influence water levels at the shoreline.

The following five tidal and oceanic cycles are the primary drivers of regular (non-climate change related) water level variations, presented in order of increasing timescale from shortest to longest:

- A mixed semidiurnal tidal cycle, with two high tides and two low tides occurring each day, with each of the four tides reaching different elevations. These astronomical tides are driven by the gravitational attraction between the rotating earth, moon, and sun, shown in Figure 18
- A 14-day spring-neap cycle, with the highest energy (and largest tide range) occurring during spring tides during the new and full moon, and the lowest energy (and smallest tide range) occurring during neap tides when the sun and moon are at right angles to each other, shown in Figure 19.
- An annual cycle in which water levels are generally lower in the spring and early summer and higher in the early fall through winter. The spring drop in sea level along the California coast, and observed at the San Francisco Presidio tide gage, results from the onset of strong winds blowing

toward the Equator. These winds, plus the Coriolis effect, push surface waters away from the coast. The higher sea level in the fall and winter are caused by the reduced alongshore winds, coupled with the expansion of water caused by summer and fall warming trends.

- The El Niño/La Niña cycle (ENSO) where every 2 to 7 years the equatorial trade winds relax, or even reverse, and warm surface water moves back along the Equator toward South America. During El Niño conditions, unusually long, low waves are generated in the Pacific Ocean called “Kelvin waves.” These waves appear to be about 6,000 miles long, traveling at speeds of more than 5 mph, with heights of up to 12 inches. As a Kelvin wave moves along the west coast, the higher water levels are propagated into the Bay.
- PDO is a long-term (for example, 20 – 30 years) ocean fluctuation of sea surface temperatures in the Pacific Ocean. PDO includes both warm and cool phases which alter upper level atmospheric winds, affecting ocean water levels, hurricane activity, droughts, and floods. The phase of the PDO can also affect the relative local rate of sea level rise. The cool phase of the PDO pushes warm water away from the U.S. Pacific coast, suppressing local sea level rise.

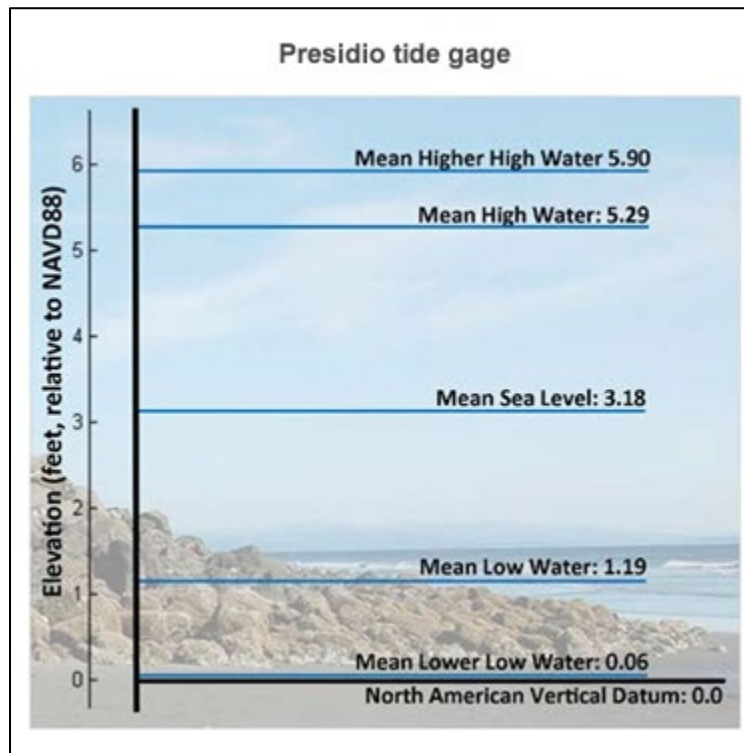
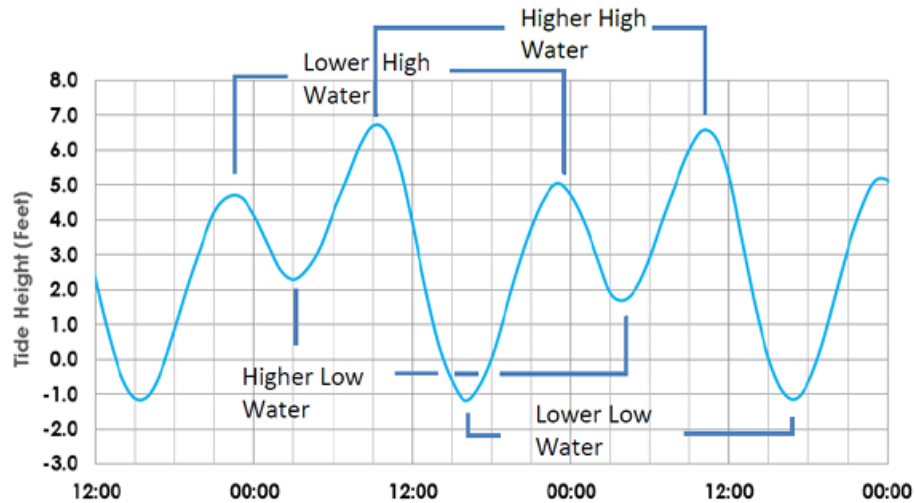


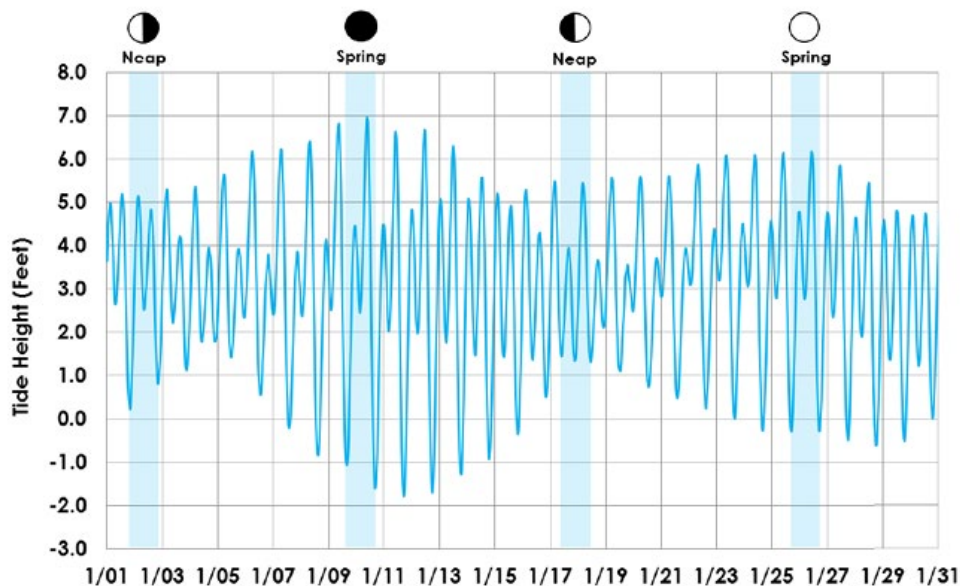
Figure 20. San Francisco (Presidio) Tidal Datums





Source: (May et al. 2016b), retrieved from NOAA Presidio tide gage

Figure 21. Daily High Tide and Low Tide Patterns in a Mixed Semidiurnal Tide



Source: (May et al. 2016b), retrieved from NOAA Presidio tide gage

Figure 22. Spring-Neap Tide Cycle

4.2.3. Current Conditions

A more in-depth assessment of the current climate and tidal conditions in the study area can be found in Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding and Appendix J: Climate of the San Francisco Waterfront Feasibility study.

4.2.3.1. Current Water Levels

For San Francisco, even the difference between the 10% and the 1% return frequency is less than 12 inches, see in Table 5. Events with a larger recurrence interval (lower return frequency), such as the 0.5% AEP or 0.2% AEP event are considered very rare and were not modeled in the coastal storm database used for the feasibility study.

Table 6 San Francisco Return Intervals for Various Periods of Record

Frequency	EY ^b	AEP	Recurrence	Presidio Water Level (feet NAVD88) ^a	
				1900 - 2020	1970 - 2020
Very Frequent	12	99.999386%	1-month	6.87	6.91
	6	99.75%	2-month	6.98	7.01
	4	98.17%	3-month	7.04	7.09
	3	95.17%	4-month	7.12	7.17
	2	86.47%	6-month	7.23	7.28
Frequent	1	63.21%	1-year	7.42	7.47
	0.5	39.35%	2-year	7.62	7.67
	0.2	18.13%	5-year	7.88	7.95
	0.1	9.52%	10-year	8.09	8.18
Rare	0.04	3.92%	25-year	8.36	8.48
	0.02	1.98%	50-year	8.57	8.73
	0.01	1.00%	100-year	8.78	8.98

^a Water Levels (feet NAVD88) are calculated for the Presidio tide gage, baselined to 2000

^b EY = Average number of exceedances per year

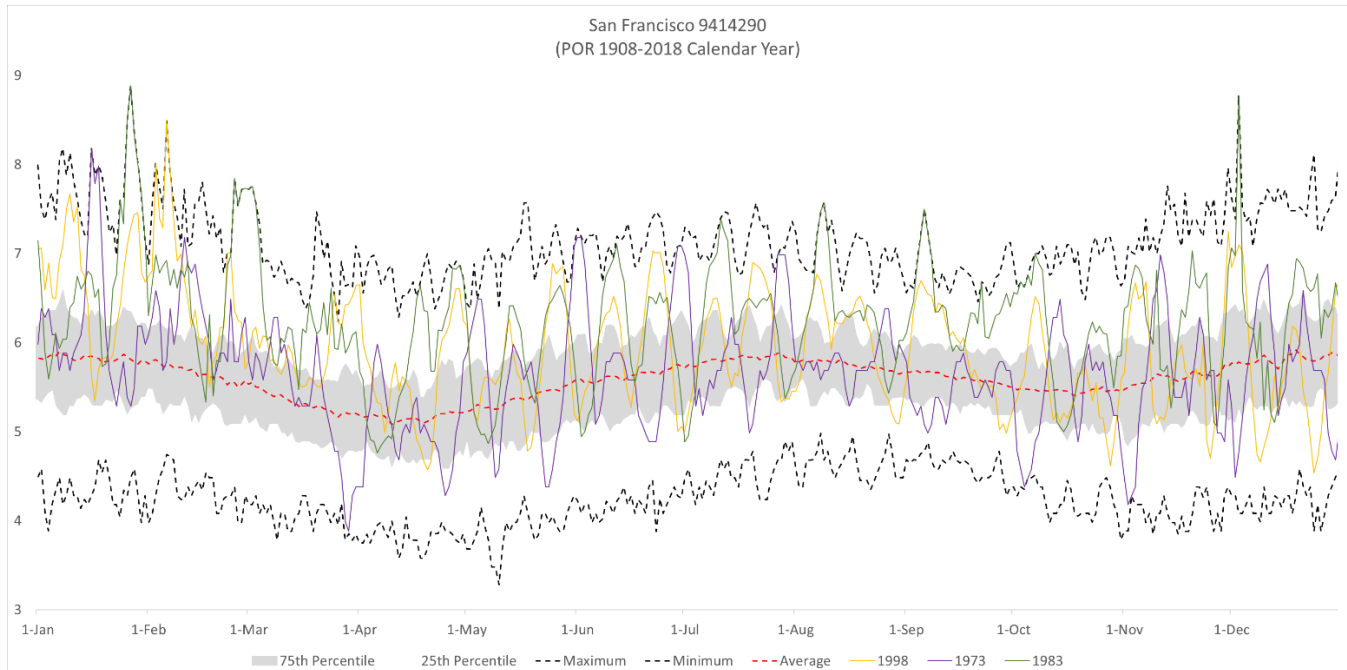


Figure 23 Summary of Peak Daily Water Level Elevation Data for San Francisco Tidal Gage

4.2.3.2. Wind Waves

The wind climate above the Bay and the larger Bay Area is highly variable, and the steep topography, hills, and valleys throughout the Bay Area drive complex local wind patterns. Due to the large size of the Bay, the winds have sufficient fetch to generate wind-driven waves that are 3 to 5 feet in height along the most exposed sections of the Bay shoreline. Due to the orientation of the San Francisco shoreline, the most impactful winds are (1) easterly (that is, offshore) winds that can impact the shoreline from the Ferry Building and southward, (2) north and northeasterly winds that can impact the northern waterfront, and (3) southeasterly winds that can impact the southern waterfront. Strong winds blowing over the Pacific Ocean can also drive longer-period ocean swell that enter through the Golden Gate. The strongest winds of the year occur during spring (that is, March, April, and May). The large storm systems that occur in winter often continue into March.

4.2.3.3. Ocean Swell

The San Francisco shoreline is also impacted by ocean-driven swell (that is, the longer-period waves that develop in the Pacific Ocean where the onshore fetch can span thousands of miles), creating damaging waves along the energetic open coast. As the storms over the Pacific Ocean begin to become stronger and more frequent by late October and into November, longer-period ocean swells begin to penetrate through the Golden Gate. The storm tracks across the Pacific and toward the west coast becomes increasingly active in November, and extratropical cyclones and atmospheric rivers begin to roll through the Bay Area. As the ocean-driven swell propagates through the narrow and deep Golden Gate channel, the swell waves continue to travel generally in the same direction. The waves quickly dissipate energy and decrease in height as they enter the Bay. The 1% AEP swell wave height is approximately 8 feet at the Golden Gate Bridge, and this wave height decreases to about 2.5 feet offshore of Aquatic Park, and to about 0.5 feet at the San Francisco-Oakland Bay Bridge.

4.2.3.4. Storm Characterizations for the San Francisco Bay

Two types of storm categories produce most of the coastal hazards and flooding in San Francisco. These are extratropical cyclones which develop offshore and can bring a range of weather patterns and atmospheric rivers which originate in the tropic primarily bring rain and wind. Both types of storms bring on high winds and heavy rains. For more information refer to Appendix B.1.1.

Appendix B.1.1 identified 15 large storms, shown in Table 6, that represent a range of climactic conditions that are most relevant to the San Francisco waterfront. The list of storms included the period of record of 1908 through 2018.

Table 7. Key Observed Historical Events and Storm Characteristics (Appendix B.1.1)

Storm Year	Storm Duration (days)	Storm Dates NOAA SF Downtown Station ^a		Storm Type	ONI ^b	Precipitation Totals SF Downtown Station ^c					Wind Speed (max hourly)		Wind Gust (instantaneous)	Min Barometric Pressure	Max Non-tidal Residual	Maximum Observed Water Level	Residual (Storm Surge) at Max Observed Water Level
		Storm Start	Storm End			Storm Total (in)	1-hr MAX (in)	3-hr MAX (in)	12-hr MAX (in)	24-hr MAX (in)	SFO (mph)	SFO (deg)	SFO (mph)	SFO (mb)	Presidio (feet)	Presidio (feet-NAVD)	Presidio (feet)
2014	5	12/2/2014	12/6/2014	cyclone	+0.7	3.4	0.8	1.2	1.6	2.3	31	180	39	1005.1	1.4	7.8	1.3
2014	2	12/11/2014	12/12/2014	AR + cyclone	+0.7	3.7	0.6	1.0	2.5	3.5	29	150	45	999.3	1.9	6.3	1.5
1982	3	1/3/1982	1/5/1982	AR + weak cyclone	0.0	4.7	0.5	1.1	2.7	4.2	30	140	35	1003.3	1.4	7.0	1.1
1994	4	11/4/1994	11/7/1994	AR	+1.0	6.9	0.7	1.9	5.0	6.2	23	360	32	1011.3	0.5	7.2	0.3
1998	9	1/31/1998	2/8/1998	AR + cyclone	+1.9	9.2	0.4	0.9	2.4	3.6	30	110	41	989.0	3.1	8.5	2.5
1995	4	12/10/1995	12/13/1995	AR + cyclone	-1.0	5.5	0.5	1.0	2.5	3.7	54	220	55	999.0	2.1	6.4	1.5
1986	11	2/11/1986	2/21/1986	AR + cyclone	-0.5	7.8	0.8	1.2	1.7	1.9	48	190	56	998.0	1.8	6.6	1.2
1983	4	1/21/1983	1/24/1983	AR + cyclone	+2.2	2.4	0.4	0.7	1.2	1.2	29	210	52	1007.1	1.6	7.6	1.4
2009	1	10/13/2009	10/13/2009	AR + cyclone	+1.0	2.5	0.5	1.1	2.4	-	41	180	56	995.2	1.6	6.6	1.1
2018	2	1/8/2018	1/9/2018	AR from south + weak cyclone	-0.9	2.6	0.1	0.4	1.3	2.4	32	280	46	1015.6	1.0	6.7	0.9
2008	6	1/22/2008	1/28/2008	cyclone from high-latitudes	-1.6	4.5	0.4	1.1	2.7	3.2	24	180	38	999.2	1.1	6.8	0.1
1990	3	2/15/1990	2/17/1990	cyclone from high-latitudes	+0.2	1.8	0.3	0.7	1.4	1.5	55	220	62	1006.1	0.9	5.7	0.2

San Francisco Waterfront Storm Damage Risk Reduction System

Storm Year	Storm Duration (days)	Storm Dates NOAA SF Downtown Station ^a		Storm Type	ONI ^b	Precipitation Totals SF Downtown Station ^c					Wind Speed (max hourly)		Wind Gust (instantaneous)	Min Barometric Pressure	Max Non-tidal Residual	Maximum Observed Water Level	Residual (Storm Surge) at Max Observed Water Level
		Storm Start	Storm End			Storm Total (in)	1-hr MAX (in)	3-hr MAX (in)	12-hr MAX (in)	24-hr MAX (in)	SFO (mph)	SFO (deg)	SFO (mph)	SFO (mb)	Presidio (feet)	Presidio (feet-NAVD)	Presidio (feet)
2004	4	2/24/2004	2/27/2004	AR + cyclone	+0.3	2.3	0.7	1.4	1.9	2.0	31	130	45	1002.2	1.9	6.2	0.8
2005	3	12/29/2005	12/31/2005	AR	-0.8	3.0	0.6	1.1	2.4	2.8	38	190	55	998.9	1.3	8.0	1.2
2017	6	1/7/2017	1/12/2017	AR + cyclone	-0.3	3.9	0.3	0.4	1.1	1.4	44	170	61	1005.8	1.4	7.7	0.7

^a Storm start/end dates and precipitation characteristics using NOAA San Francisco Downtown Station (COOP: 047772; 1980 – 2011); SFPUC Mission Street Station (ID: RG31; 2011 – 2018).

^b ONI is a 3-month running mean of sea surface temperature anomalies in the Niño 3.4 region (of the Pacific Ocean). This is a measure used to define El Niño or La Niña events. ONI between 1.0 and 1.5 is classified as moderate; between 1.5 and 2.0 as strong; and above 2.0 as very strong.

^c Precipitation related characteristics are not relevant for the G2CRM coastal storm inputs; however, they are nevertheless helpful in communicating the importance of considering precipitation-related flooding and impacts outside of G2CRM.

AR = atmospheric river
deg = degree(s)
in = inch(es)
mb = millibars
ONI = Oceanic Niño Index

Two storms were included in more detail, one of which occurred in December 2022 – January 2023 which was not available when the list of storms was completed.

December 2 – 12, 2014

The first 2 weeks in December 2014 marked one of the wettest periods on record for the Bay Area. This series of storms resulted in widespread flooding, street closures, and power outages in San Francisco. The storms originated as a weak extratropical disturbance, but eventually merged with a low-pressure system from the Aleutian Islands and strengthened while the high-pressure system off the California coast dissipated. This resulted in the atmospheric river that reached the Bay Area between December 9 and 12.

On December 3, precipitation from a low-pressure cyclone drenched downtown San Francisco with 2.3 inches of precipitation within 24 hours. Peak tide at San Francisco gage for this event was 7.8 feet. Maximum wind speeds at the airport reached 31 mph from the south, and a maximum wind gust of 39 mph was recorded. The low-pressure system also brought high tides with almost 1.5 feet of storm surge on top of the daily high tides. Several neighborhoods experienced flooding that are outside of the SFWRFS study area. The most severe flooding was near 15th and Wawona Street and in the 17th and Folsom neighborhood.

The storm was quickly followed by an atmospheric river that arrived on December 11 and delivered 3.4 inches of precipitation. Peak tide at San Francisco gage for this event was 6.3 feet. Coupled with 1.5 feet of storm surge, San Francisco was flooded from both precipitation runoff and high Bay waters that overtopped the shoreline. High windspeeds of 29 mph from the south-southeast were recorded at the airport, with maximum wind gusts of 45 mph. The December 11 storm resulted in more severe widespread flooding, street closures, and power outages across the Bay Area than the earlier December storm.

December 2022 - January 2023

Between December 26, 2022 and January 16, 2023, the Bay Area was hit with nine consecutive ARs, some that co-occurred with ETCs, and one that became a powerful bomb cyclone on January 5, 2023. In

San Francisco, 18 inches of rain fell over the 21 days, representing 75% of the total average annual rainfall. The largest daily rainfall total was 5.14 inches on 31 December 2022 and the peak tide for that time period was 7.74 feet on 05 January 2023.

The 2022-2023 winter season started out as one of the wettest winters on record, even though it was projected to be a lower precipitation year due to the La Niña conditions, breaking a 152-year-old record for the second wettest 10-day period since 1871. The wettest 10-days on record occurred in 1862.

4.3. Historic Trends and Projections

Appendix J Climate and Appendix B.1 provide a range of information in regard to historic and projected trends in regard to sea level change, precipitation, climatic cycles, temperatures, etc. The following sections provide a quick summary of the perception and sea level change information. Refer to the Appendices for more detailed discussion.

4.3.1.1. Precipitation

The USACE Climate Hydrology Assessment Tool was used in assessing hydrologic based climate change impacts, including evaluating past (observed) and potential future (projected) changes to relevant hydrologic inputs such as precipitation. Figure 21 displays the observed and projected trends for the annual-maximum 1-day precipitation, based on 93 different climate change hydrologic simulations for the period of 1950-2099 and assuming the RCP4.5 climate scenario. Year-to-year variability in the projected trend is expected, consistent with observations. The Bay area topography can influence both the temporal and spatial differences in local precipitation, which can be overlooked by global climate models. For this reason, several San Francisco departments partnered with Lawrence Berkeley National Laboratory and Pathways Climate Institute to conduct a regional climate modeling to better understand how Bay Area storms could change under a warming climate. Figure 22 shows the results of the historical and projected Return Period for the Rainfall Depth for 24-hour Duration.

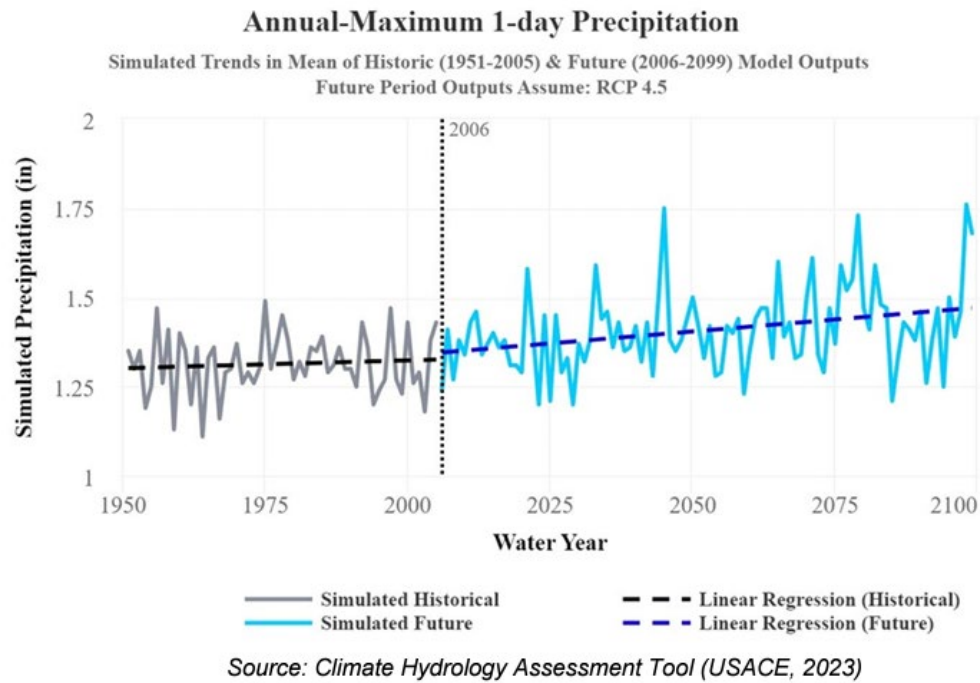


Figure 24. Observed and Projected Annual Maximum 1-Day Precipitation USACE Climate Hydrology Tool

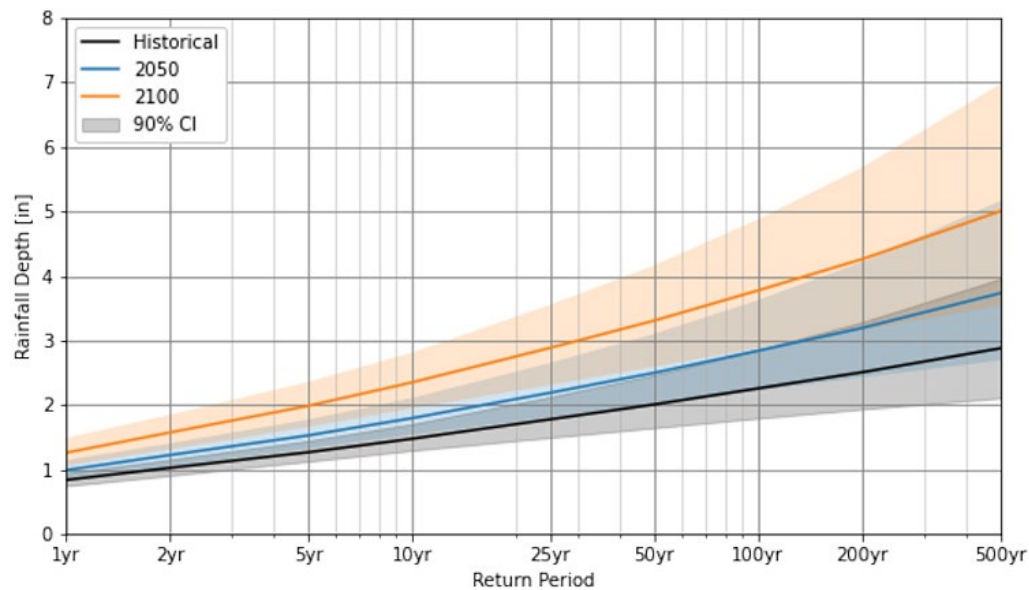
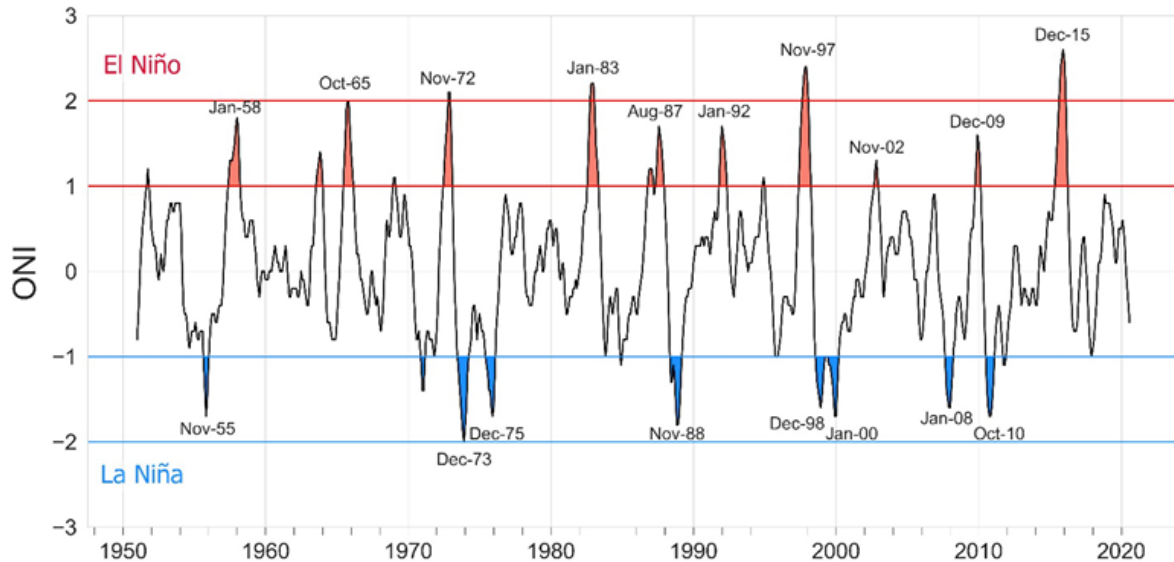


Figure 25. Historical and Projected Return Period vs Rainfall Depth for 24-hour Duration – Regional Study

4.3.2. Oceanic Niño Index

The largest observed water levels generally occur during El Niño years. Figure 23 presents the Oceanic Niño Index which measures the relative strength of the El Niño and La Niña conditions. The strength of El Niño and La Niña conditions, as measured by the Oceanic Niño Index, is also observed to increase over time.

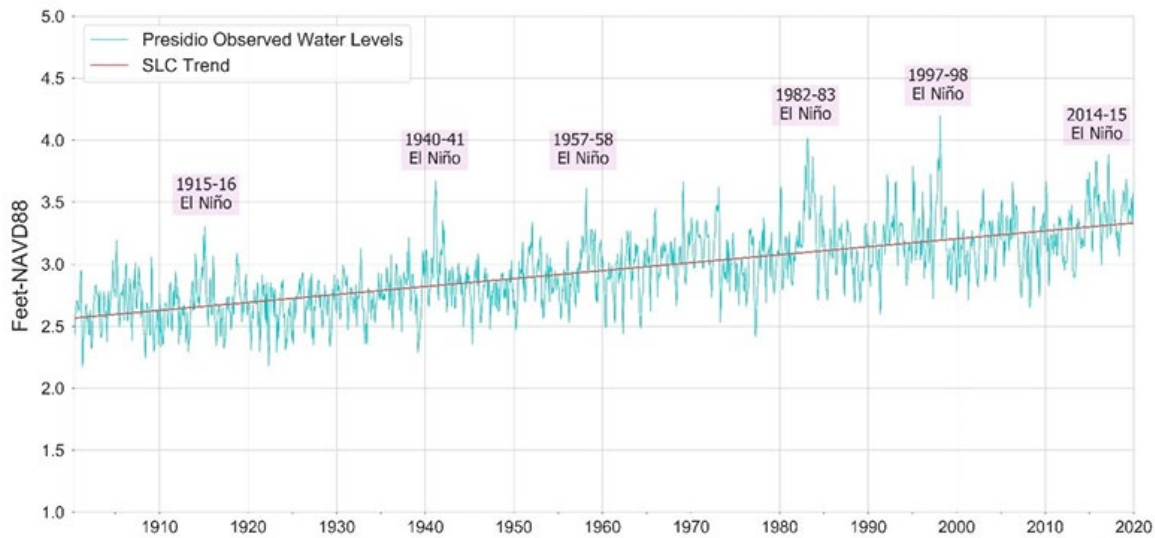


Source: (NOAA, 2023c)

Figure 26. Oceanic Niño Index

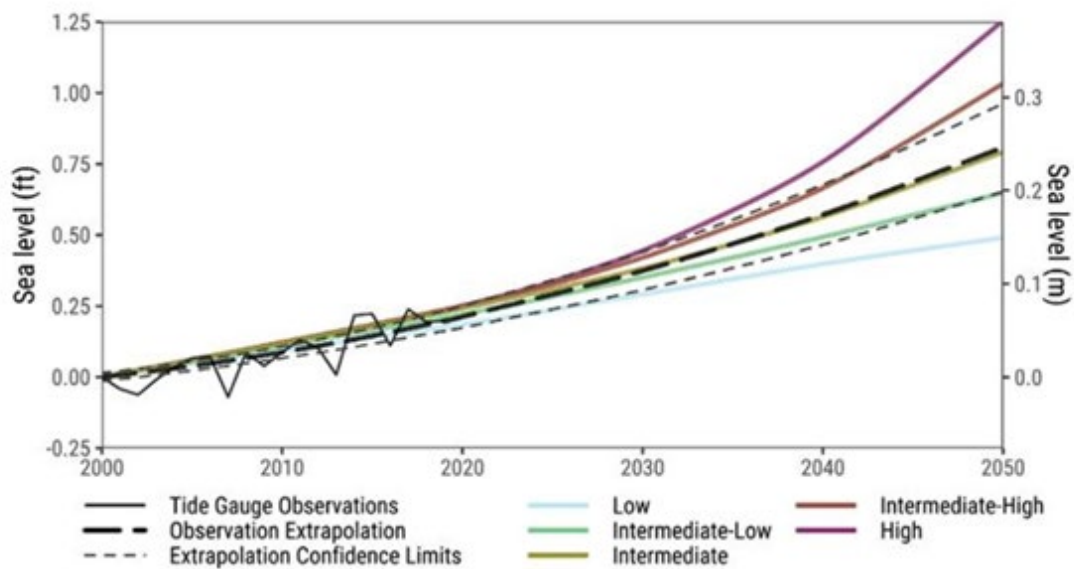
4.3.3. Sea Level Change Curves

The three USACE Sea level Change Curves were used to evaluate the feasibility study. In addition, the three USACE curves, two California Sea Level Change Curves (OPC) were in the study as references to the regional analysis that has been completed. Figure 26 shows the five curves plotted together using 2000 as the baseline year. Figure 24 shows the sea level change trend at the San Francisco Gage through 2020 relative to mean sea level. Figure 25 shows the 5 year average observed sea level change compared to the five sea level change curves.



Source: (NOAA, 2020)

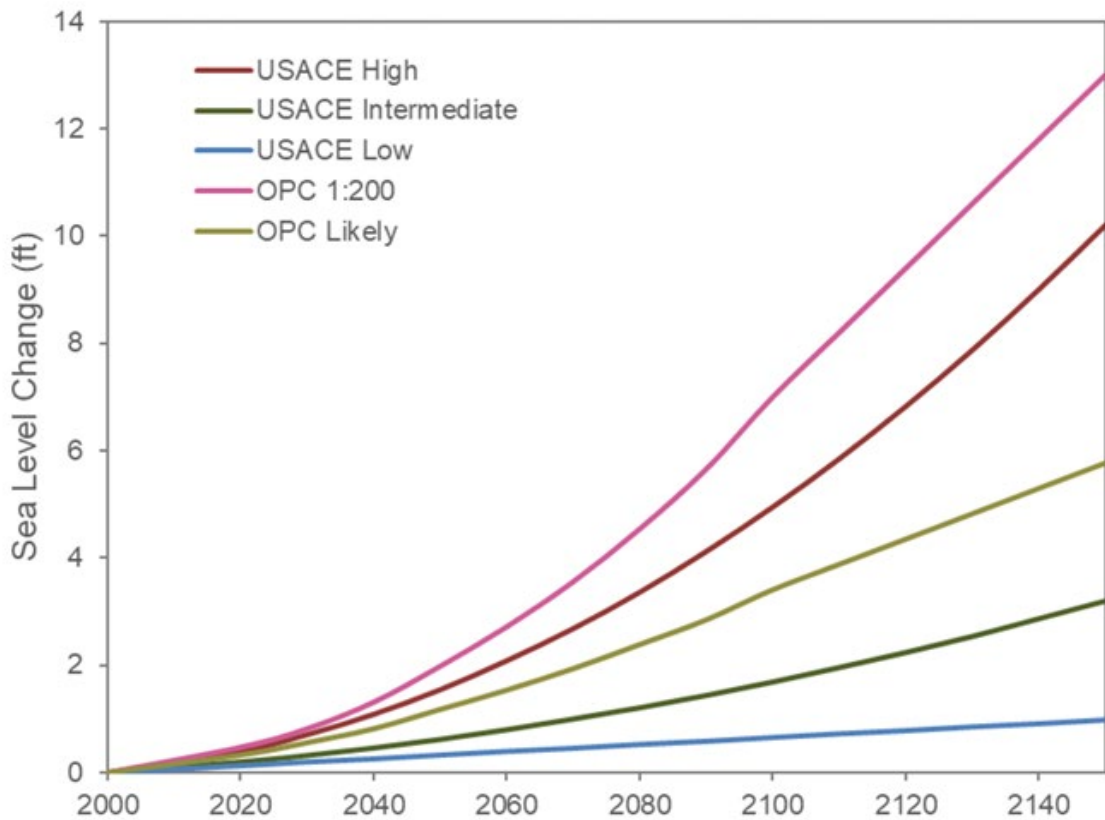
Figure 27. San Francisco Presidio Tide Gage Sea Level Record (Relative to Mean Sea Level)



Source: Sweet et al. (2022), Collini et al. (2022)

Average annual water levels from tide gages are overlaid for context. The extrapolation confidence limits represent the 17th and 83rd percentile confidence interval for the observation-based extrapolations.

Figure 28. Observed Sea Levels at San Francisco Compared to the Various SLC Curves



Source: OPC & CNRA (2018), USACE (2020)

Figure 29. USACE and State of California Sea Level Change Curves

4.4. Coastal Conditions

4.4.1. Stage-Duration Curves

Annual and Monthly stage-duration curves were developed at the San Francisco (Presidio) Tidal Gage which is representative of the primary tidal gage near the study area for the proposed new lines of defense. These duration curves were developed using HEC-DSSvue and shown in Figure 27.

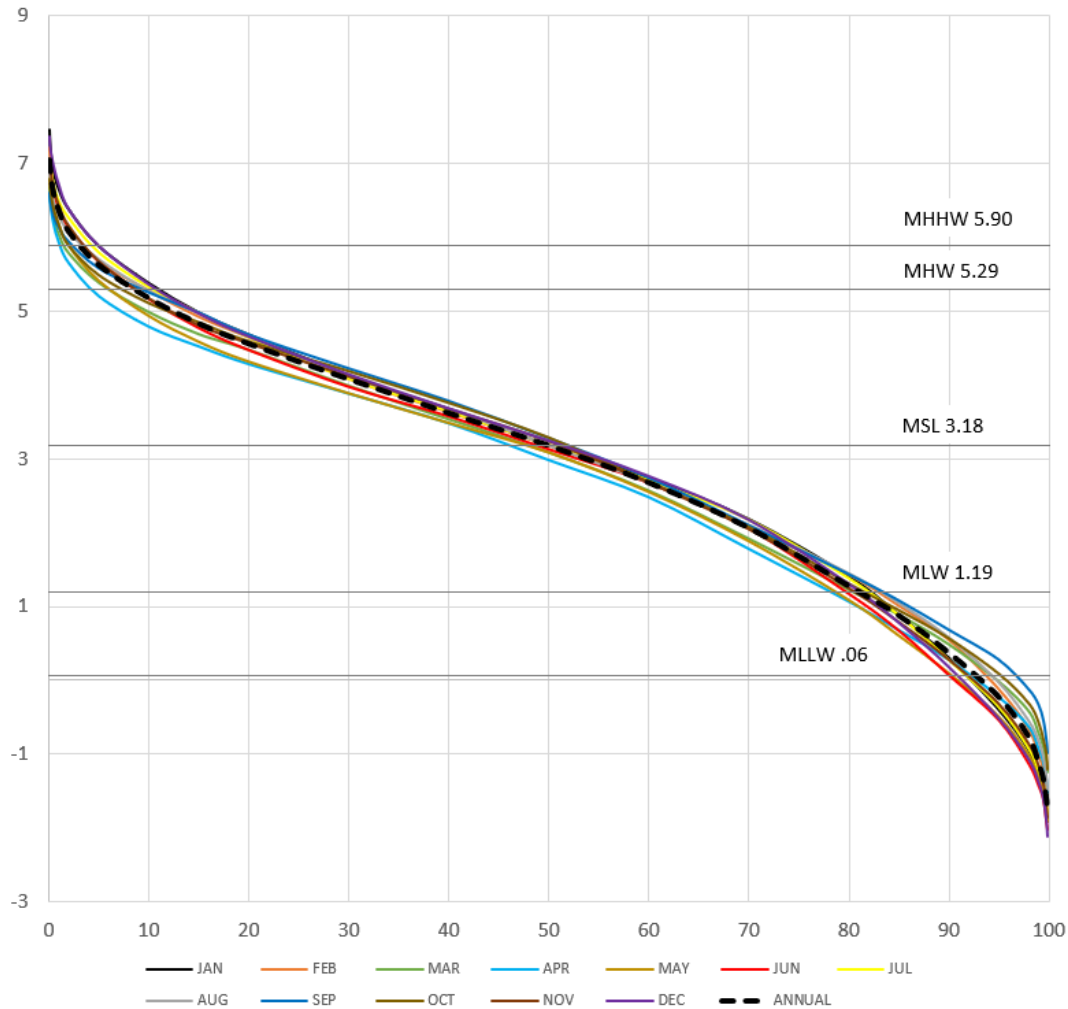


Figure 30. Stage duration curve at San Francisco 9414290 (POR 1907-2018)

4.4.2. Stage-Frequency Curves

Stage frequency values were developed for the feasibility study and are described in Appendix B.1.1. The stage frequency values were developed for each Reach in the study area using the FEMA model output. The values used for the assessment were relative to 1992 to match the USACE SLC curves and shown in Table 7 and plotted in Figure 28.

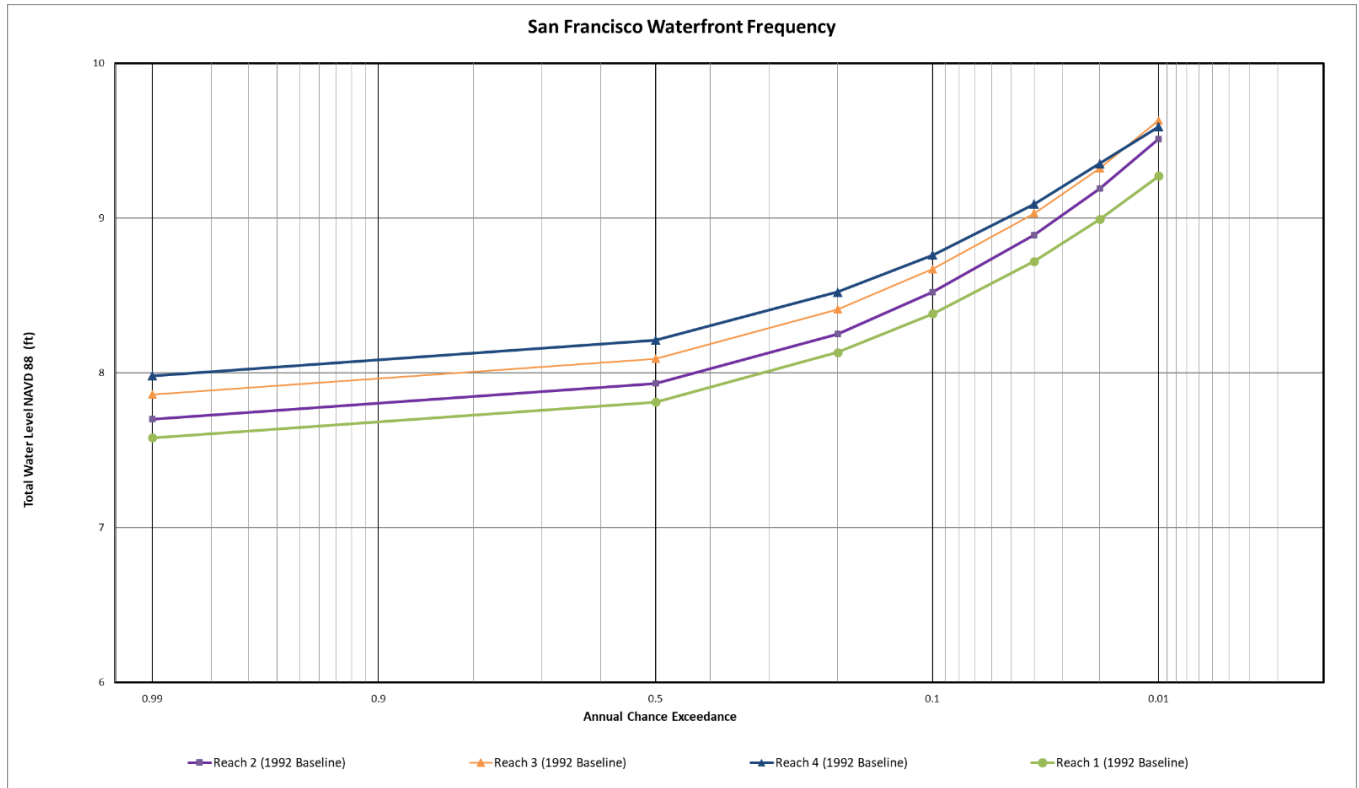
Table 8. Low Frequency AEP Values per Model Area (relative to 1992)

Model Area	1-year (63.2%)	2-year (39.3%)	5-year (18.1%)	10-year (9.5%)	25-year (3.9%)	50-year (2%)	100-year (1%) ^a
1	7.58 ^b	7.81	8.13	8.38	8.72	8.99	9.27
2	7.70	7.93	8.25	8.52	8.89	9.19	9.51
3	7.86	8.09	8.41	8.67	9.03	9.32	9.63
4	7.98	8.21	8.52	8.76	9.09	9.34	9.59

^a 1% AEP were developed using a scaled synthetic hydrograph.

^b Values are presented to two significant digits for comparison purposes, although the accuracy of the model output and analysis results do not include this level of precision.

^c Values presented do not include the contribution of wind driven waves.

**Figure 31. Total Water Level Frequency Curve for Reaches 1-4 (1992 Baseline)**

4.5. Hydraulics

4.5.1. *Hydraulic Model*

The two-dimensional (2D) model developed for the interior drainage analysis for the feasibility study was modified to complete the breach analysis for the risk assessment. The geometry is split into two separate 2D areas, one for the area that is primarily served by the northern water treatment plant and one for the area that is primarily served by the southern water treatment plant. This is done so simplified combine storm sewer operations could be incorporated. The 2D areas have cells that range from 40 feet to 160 feet, with the majority of the area at 120 feet. The smaller cell sizes are enforced at the breaklines. Light Detection and Ranging (LiDAR) data is available for the entire San Francisco area. The digital elevation model (DEM) generated from the 2010 LiDAR survey at a 1 foot resolution was used for the interior drainage analysis and carried forward for the H&H breach analysis for the Risk Assessment. Future developments provided by the Port of San Francisco were also incorporated into the terrain. The Land Cover layer used in the CCSF19 ICM model are used for the HEC-RAS model. This layer includes all existing conditions, green infrastructure implemented through the Stormwater Management Ordinance from 2010 through 2032. The Second Action Geometry is shown in Figure 29 and shows the general layout for the 2D elements and terrain. A separate geometry is used for the First Action and Alternative G, as they have different alignments for the line of defense.

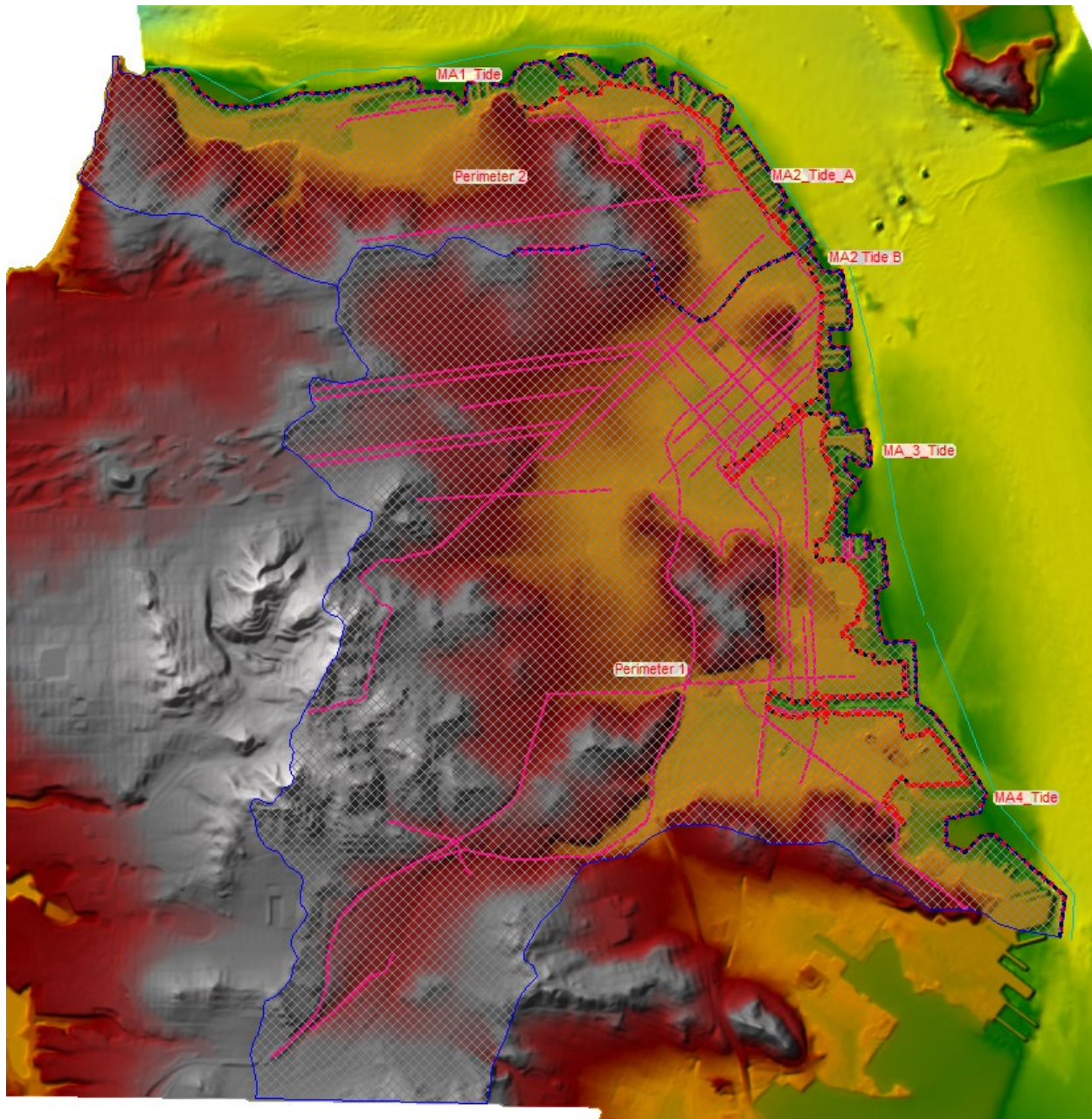


Figure 32. HEC-RAS Geometry Example (Second Action)

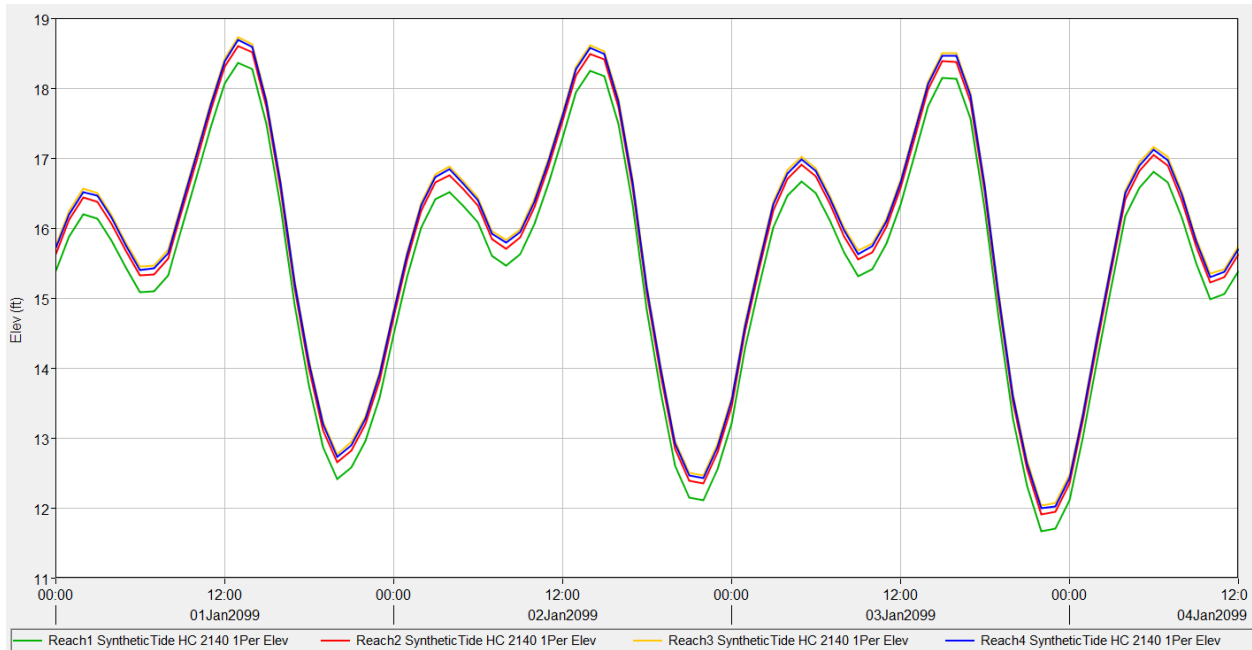
4.5.2. Boundary Conditions

For the exterior boundary condition, a three-day observed tide at Presidio was used to mimic a synthetic tide. This synthetic tide was scaled to match the 1% AEP total water levels based on the 1992 baseline condition. The sea level change condition from the USACE Intermediate Curve was applied to the 2090 First Action line of defense and the USACE High Curve 2140 Second Action line of defense. Table 8 shows the peak total water level for each of the reaches and Figure xx shows a plot of the exterior condition for 2140.

Table 9 Summary of Peak Total Water Level First and Second Action Scenarios

Model Area	1% AEP from Coastal Report (NAVD 88)	Intermediate Curve 2090 (NAVD 88)	High Curve 2140 (NAVD 88)
Sea Level Change (feet)	N/A	1.5	9.1
Reach 1	9.27	10.77	18.37
Reach 2	9.51	11.01	18.61
Reach 3	9.63	11.13	18.73
Reach 4	9.59	11.09	18.69

Note. 1% AEP values are based on 1992 baseline conditions.

**Figure 33. 2140 Exterior boundary condition for Reaches 1-4**

4.5.3. Levee System Profile

An estimated system profile for the proposed line of defense for the first and second action for the tentatively selected plan was developed in Microsoft Excel using estimated profiles from the terrain file and stationing for the system. The stationing does not represent the exact station but follows the general line of defense and high ground for the two alternatives. Total water levels for the 99%, 10% and 1% with

the sea level rise adjustments for 2090 USACE intermediate curve and 2140 USACE high curve and general reach delineations.

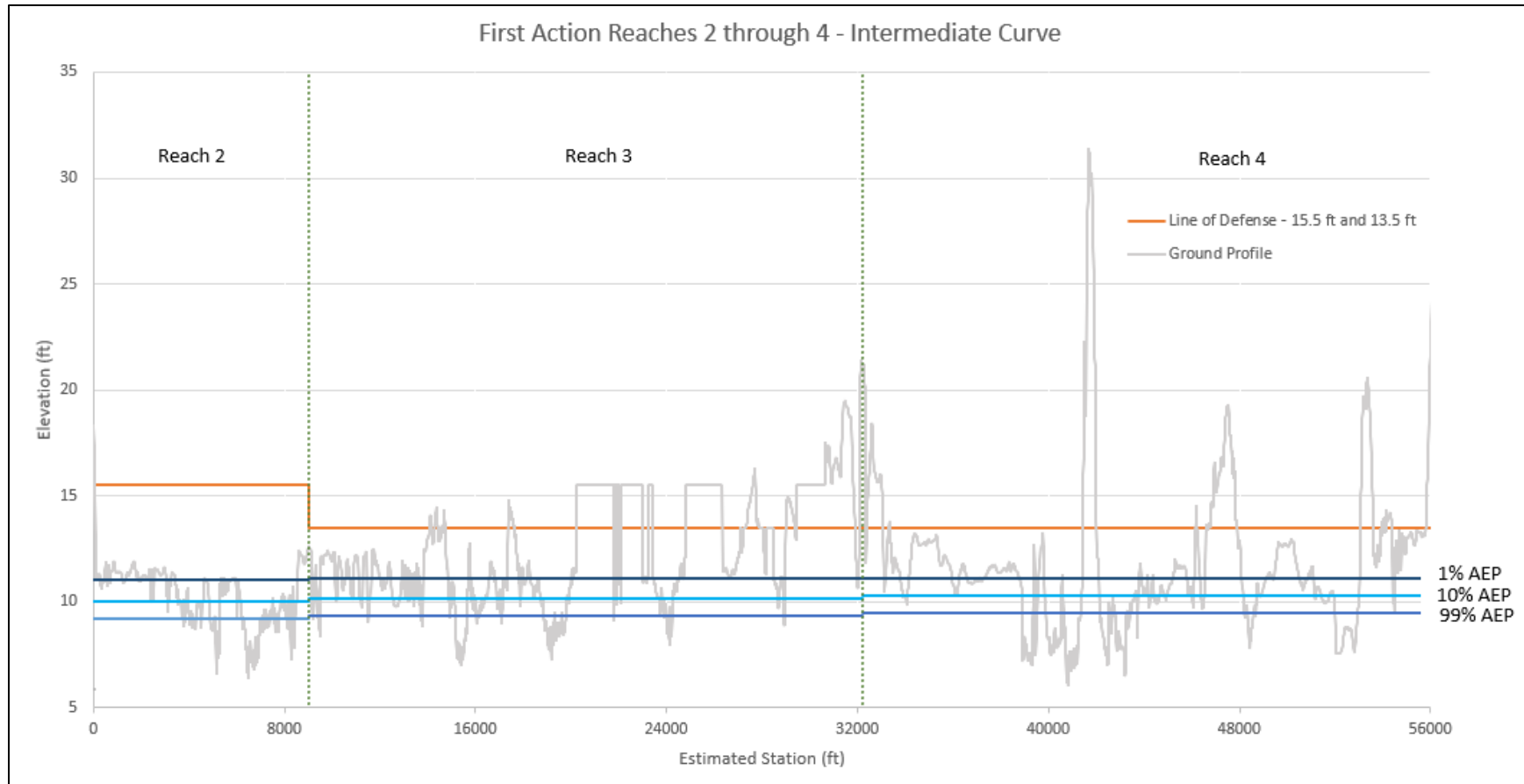


Figure 34. General Profile for the First Action Line of Defense with various Water Levels with Sea Level Rise at 2090 with the Intermediate Curve

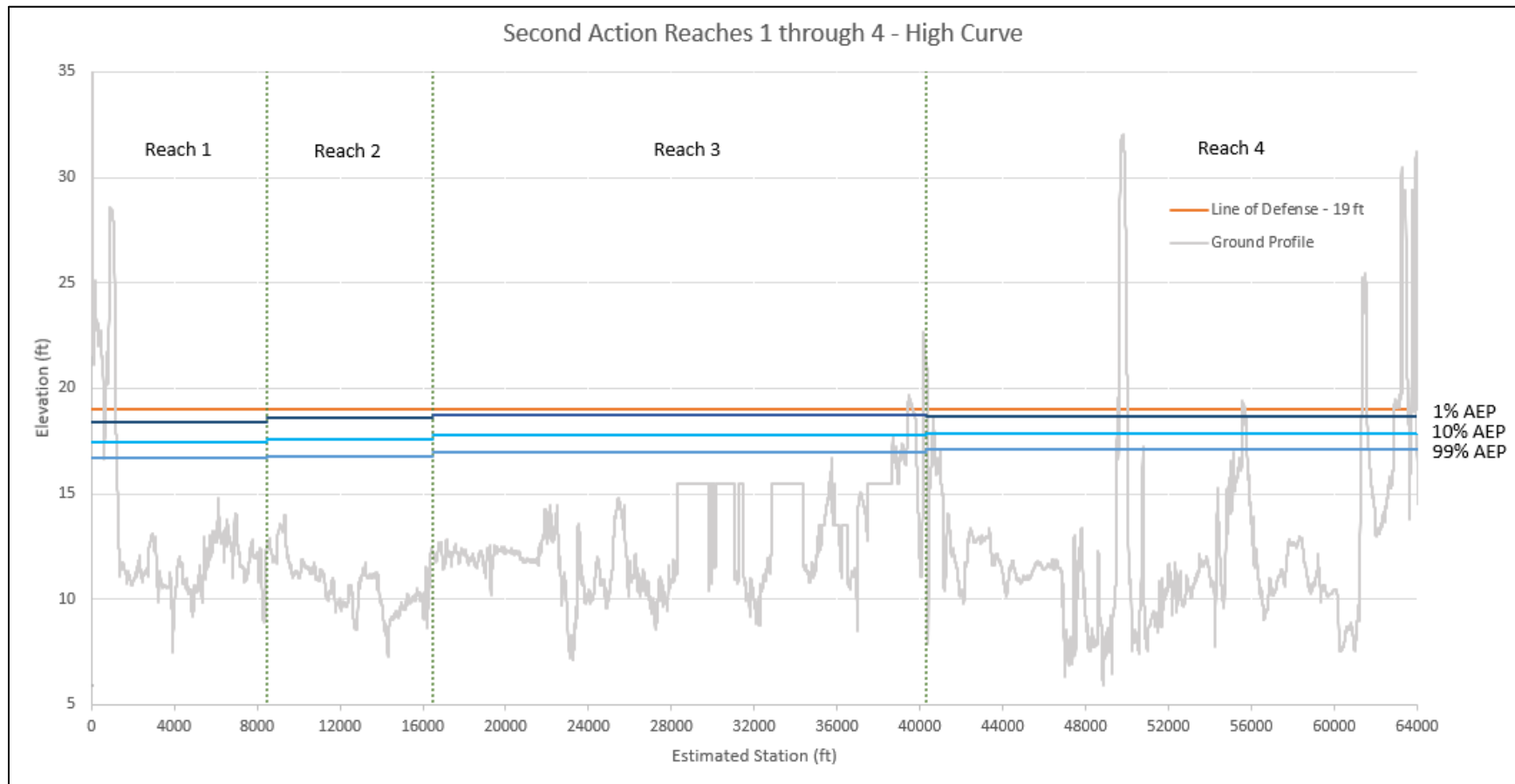


Figure 35 General Profile for the Second Action Line of Defense with various Water Levels with Sea Level Rise at 2140 with the High Curve

4.6. Breach Analysis

A simplified breach analysis was completed for the First and Second Action TSP. In addition, sensitivity runs were completed using the Alternative G line of defense with tidal loading for 2140 for Reaches 3 and 4 and adding blocked for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave. in Reach 4 for both the TSP and Alternative G with tidal loading from 2140.

4.6.1. *Breach Locations*

Multiple breach locations were identified as points of vulnerability on the shoreline and the population behind the structure. The breach locations for the First Action TSP, Second Action TSP, and Alternative G are show in Figure 33 through Figure 35.

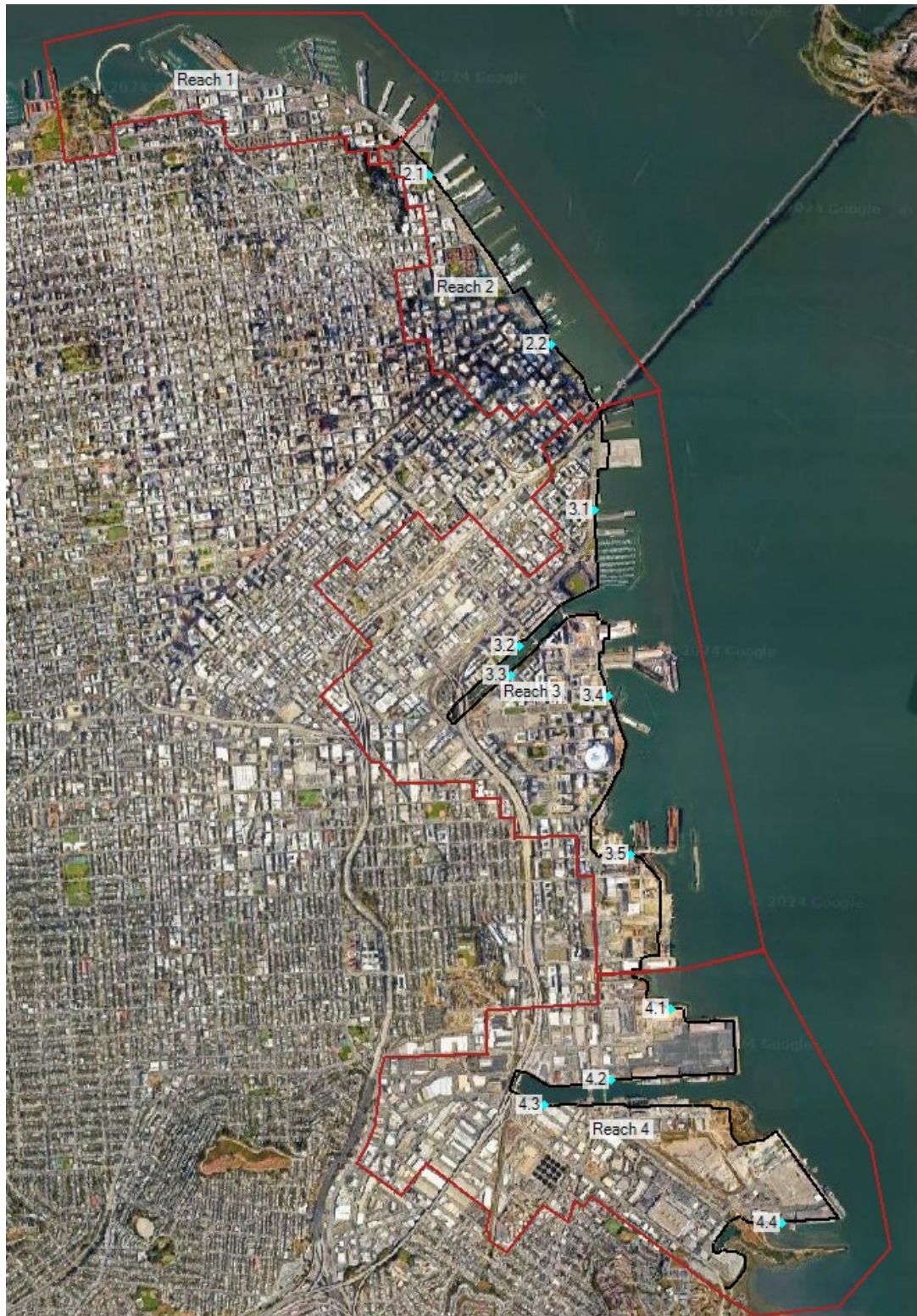


Figure 36. First Action TSP Approximate Breach Locations

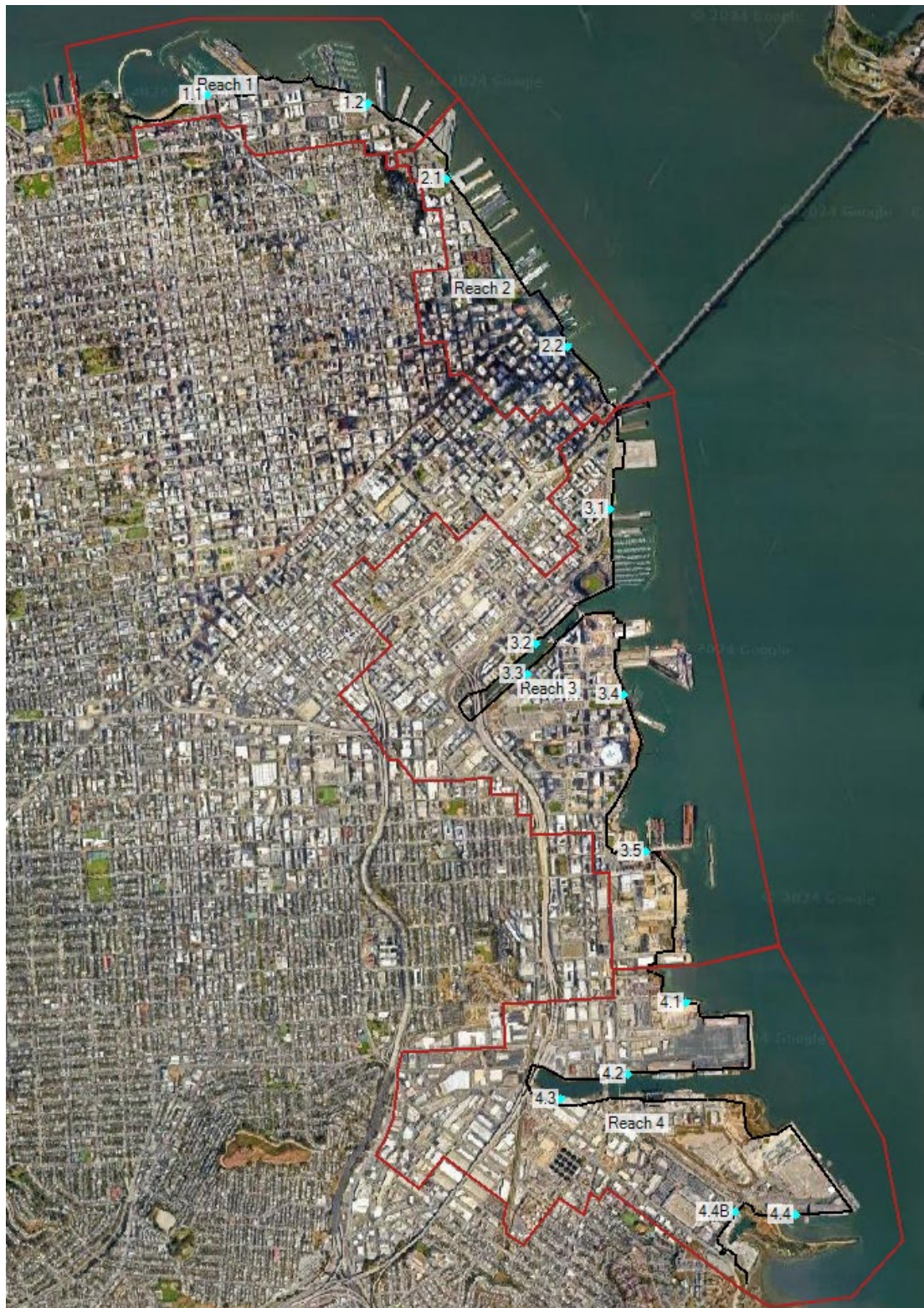


Figure 37. Second Action TSP Approximate Breach Locations



Figure 38. Alternative G Reaches 3 and 4 Approximate Breach Locations

4.6.2. Breach Estimates

The MMC Levee Breach SOP indicates that breach widths for earthen structures are defined by the natural ground profiles and widths are no more than what would be limited by the natural ground topography. In the case of the San Francisco shoreline, there has been substantial changes made to the natural ground in areas. To estimate the breach widths for earthen levee areas the Von Thun and Gillette dam breach parameters were used. In most cases, the depth of the breach for the structures was greater than 12 ft which, was larger than the shortest dam used to develop the Von Thun parameters.

The locations where walls are proposed, such as in Reaches 1 and 2, 20 feet was used for the breach width. This accounts for 2 wall sections.

A summary of breach parameters is provided for the 2090 and 2140 tidal loadings are listed in Table 9 and Table 10.

Table 10. First Action 2090 Tidal Loading Breach Parameters

First Action - 2090					
Breach Name	Alternative	Type	Breach Top Width ft	Development Time hrs	Side Slope
1.1	TSP	Non Structural	N/A	N/A	N/A
1.2	TSP	Non Structural	N/A	N/A	N/A
2.1	TSP	Wall	20	0.5	0
2.2	TSP	Wall	20	0.5	0
3.1	TSP	Wall	20	0.5	0
3.2	TSP	Earthen Levee	194	0.4	0.5
3.3	TSP	Earthen Levee	188	0.3	0.5
3.4	TSP	Earthen Levee	195	0.4	0.5
3.5	TSP	Earthen Levee	194	0.4	0.5
4.1	TSP	Earthen Levee	181	0.3	0.5
4.2	TSP	Earthen Levee	189	0.4	0.5
4.3	TSP	Earthen Levee	187	0.3	0.5
4.4	TSP	Wall	20	0.5	0

Table 11. Second Action 2140 Tidal Loading Breach Parameters

Second Action - 2140					
Breach Name	Alternative	Type	Breach Top Width ft	Development Time hrs	Side Slope
1.1	TSP	Wall	20	0.5	0
1.2	TSP	Wall	20	0.5	0
2.1	TSP	Wall	20	0.5	0
2.2	TSP	Wall	20	0.5	0
3.1	TSP	Earthen Levee	197	0.4	0.5
3.2	TSP	Earthen Levee	210	0.5	0.5
3.3	TSP	Earthen Levee	204	0.5	0.5
3.4	TSP	Earthen Levee	211	0.6	0.5
3.5	TSP	Earthen Levee	210	0.5	0.5
4.1	TSP	Earthen Levee	198	0.4	0.5
4.2	TSP	Earthen Levee	206	0.5	0.5
4.3	TSP	Earthen Levee	203	0.5	0.5
4.4	TSP	Wall	20	0.5	0
4.4B	TSP	Earthen Levee	193	0.4	0.5
G3.1	Alternative G	Earthen Levee	202	0.4	0.5
G3.2	Alternative G	Earthen Levee	191	0.4	0.5
G3.3	Alternative G	Earthen Levee	190	0.4	0.5
G4.1	Alternative G	Earthen Levee	194	0.4	0.5

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G4.2	Alternative G	Earthen Levee	195	0.4	0.5
G4.3	Alternative G	Earthen Levee	193	0.4	0.5

4.6.3. HEC-RAS Scenarios

A list of the HEC-RAS Scenarios run for the LifeSim analysis are described below in

Table 11.

Table 12. Summary of HEC-RAS Model Scenarios evaluated in LifeSim

Plan Name	Plan Number	Breach	Tidal Boundary Condition
Breach_1.1_SynTide_1Per_HC_2140	.p10	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_1.2_SynTide_1Per_HC_2140	.p12	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_2.1_SynTide_1Per_HC_2140	.p11	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_2.2_SynTide_1Per_HC_2140	.p21	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_3.1_SynTide_1Per_HC_2140	.p09	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_3.2_SynTide_1Per_HC_2140	.p14	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_3.3_SynTide_1Per_HC_2140	.p22	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise

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			based on the high curve in 2140
Breach_3.4_SynTide_1Per_HC_2140	.p23	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_3.5_SynTide_1Per_HC_2140	.p24	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.1_SynTide_1Per_HC_2140	.p25	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.2_SynTide_1Per_HC_2140	.p15	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.3_SynTide_1Per_HC_2140	.p17	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.4_SynTide_1Per_HC_2140	.p20	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.4b_SynTide_1Per_HC_2140	.p19	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140

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Breach_2.1_SynTide_1Per_IC_2090	.p26	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_2.2_SynTide_1Per_IC_2090	.p27	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_3.1_SynTide_1Per_IC_2090	.p28	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_3.2_SynTide_1Per_IC_2090	.p29	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_3.3_SynTide_1Per_IC_2090	.p30	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_3.4_SynTide_1Per_IC_2090	.p32	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_3.5_SynTide_1Per_IC_2090	.p33	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_4.1_SynTide_1Per_IC_2090	.p34	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal

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			level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_4.2_SynTide_1Per_IC_2090	.p36	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_4.3_SynTide_1Per_IC_2090	.p37	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_4.4_SynTide_1Per_IC_2090	.p38	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the intermediate curve in 2090
Breach_1.1_SynTide_1Per_HC_G_2140	.p01	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_1.2_SynTide_1Per_HC_G_2140	.p02	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_2.1_SynTide_1Per_HC_G_2140	.p03	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_2.2_SynTide_1Per_HC_G_2140	.p07	20 ft breach at on wall	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise

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			based on the high curve in 2140
Breach_G3.1_SynTide_1Per_HC_G_2140	.p42	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G3.2_SynTide_1Per_HC_G_2140	.p44	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G3.3_SynTide_1Per_HC_G_2140	.p46	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G4.1_SynTide_1Per_HC_G_2140	.p48	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G4.2_SynTide_1Per_HC_G_2140	.p50	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G4.3_SynTide_1Per_HC_G_2140	.p52	breach based on Von Thun equations	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.2_SynTide_1Per_BlockR4_HC_2140	.p55	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140

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Breach_4.3_SynTide_BlockR4_HC_2140	.p58	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.4_SynTide_BlockR4_HC_2140	.p61	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_4.4b_SynTide_BlockR4_HC_2140	.p60	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G4.2_SynTide_BlockR4_HC_G_2140	.p57	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140
Breach_G4.3_SynTide_BlockR4_HC_G_2140	.p59	Same as previous run, with block openings for Roads and Pedestrian walkways between Cesar Chavez and Evans Ave.	Synthetic tide based on an actual event and shifted based on the peak tidal level of the 1 Percent elevation with sea level rise based on the high curve in 2140

4.7. Supporting Analyses

4.7.1. *Wind-wave estimates*

Wind-wave estimates were not included in this assessment. More information on the wave overtopping sensitivity can be found in Appendix B.1.3.

4.7.2. *Overtopping/overflow velocities and durations*

Overtopping was not evaluated.

4.7.3. *Interior drainage and ponding*

Pumps for interior drainage and ponding was not evaluated. Additional large pumps are included in the feasibility study but were not considered in the assessment.

4.8. Major uncertainties

Waves were not included in the analysis. This could lead to overwash volumes not accounted for in the current analysis.

The pumps included for the interior drainage, both present day and proposed, were not included in the analysis.

Breach estimates were based on Von Thun & Gillette which is primarily used for Dams. The Tulsa and West Tulsa erosion estimates presented in the SQRA report were also evaluated but not carried forward in the assessment since it was based on a sandy soil which is highly erodible and not consistent with the conditions in San Francisco. These estimates provided a range to work with but would need to be refined in the future.

None of the combined storm sewer system features, ie CSOs, proposed culverts with flap gates, are included in the analysis.

5. Seismic Hazards

5.1. Background

The project is located at latitude 37.796°, longitude -122.393° in the central and eastern western United States. The 2018 update of the National Seismic Hazard Model (NSHM) by the U.S. Geological Survey (USGS) (Petersen et al. 2020) provides consensus-based regional analyses of seismic hazard, using Central and Eastern United States (CEUS) seismic sources, Western United States (WUS) seismic sources, combined geologic/geodetic information, and revised ground-motion models. This report represents the most current assessment of seismic hazard for large regions of North America. The 2018 NSHM declustered seismicity catalog is shown in Figure 36.

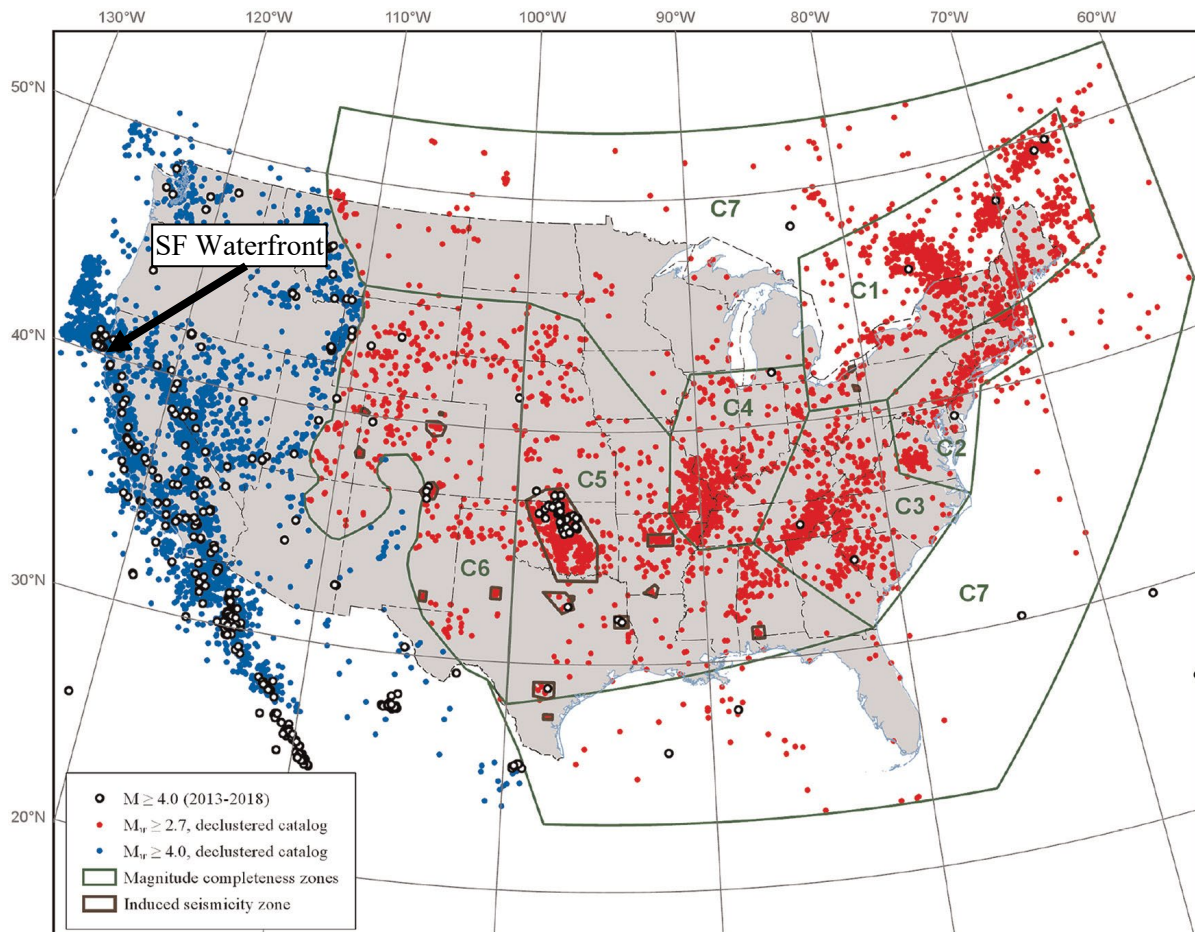


Figure 39. 2018 NSHM declustered seismicity catalogs for $M_w \geq 2.7$ (Petersen et al. 2020)

The USGS Quaternary Fault and Fold Database of the United States and interactive fault map (<https://doi.org/10.5066/F7S75FJM>) were reviewed for faults and associated folds in the vicinity of the site that demonstrate geological evidence of coseismic surface deformation in large earthquakes during the past 2.58 million years (Myr). The results are portrayed in Figure 38 and summarized in Table 12.

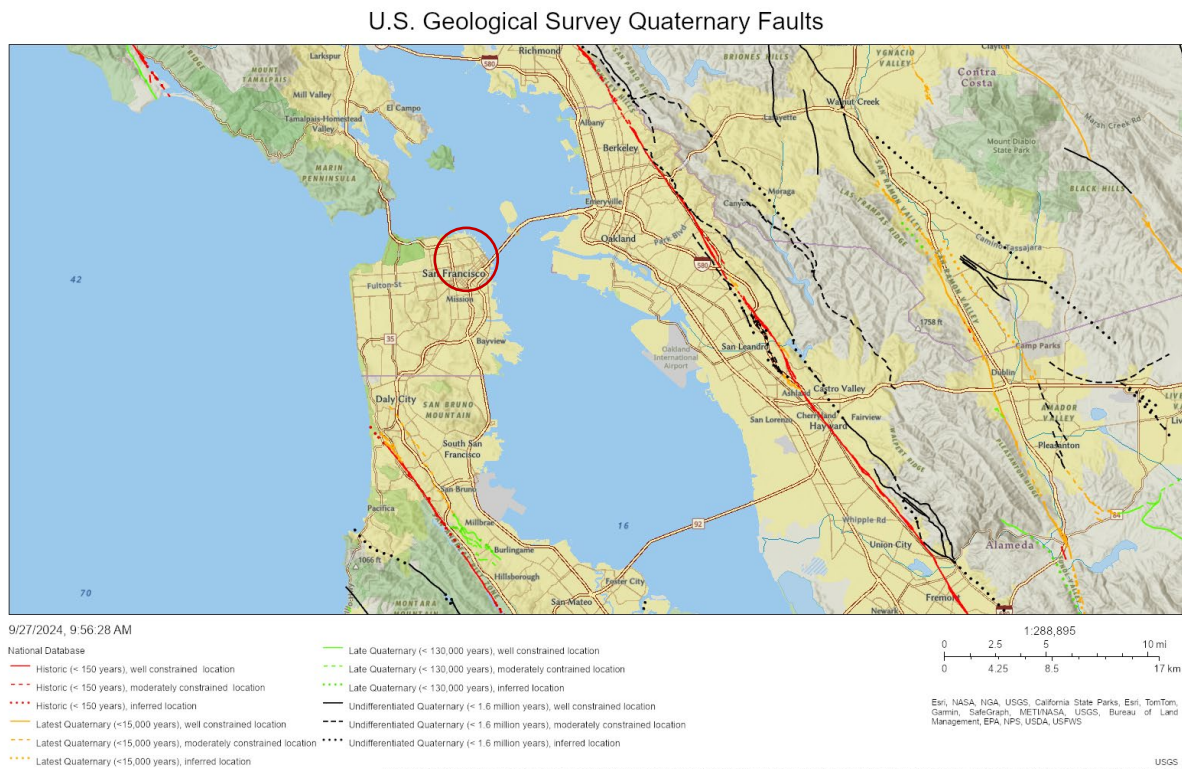


Figure 40. Quaternary faults (USGS 2024)

Table 13.
Quaternary faults (USGS).

Name	Location	Length (km)	Historic earthquake	Recent prehistoric Deformation	Slip-Rate Category
San Andreas fault zone	San Francisco, CA	1,082	San Francisco earthquake 1906	Latest Quaternary (<15 ka)	Greater than 5 mm/yr
Hayward fault zone	Oakland, CA	107	-	Latest Quaternary (<15 ka)	Greater than 5 mm/yr

NOTE: <https://doi.org/10.5066/F7S75FJM> accessed on 27 Sep 2024.

5.2. Historical Seismicity

The USGS Earthquake Catalog was searched for events with magnitude (M) greater than or equal to M4.5. Table 13 summarizes these events with respect to magnitude and epicentral distance. Table 14 provides details of some selected earthquakes of interest.

Table 14.
Historical seismicity since 1 Jan 1800 (USGS).

Magnitude Interval	Search Radius from Site	Number of Events
$4.5 \leq M < 5$	10 mi (16 km)	2
$5 \leq M < 6$	50 mi (80 km)	14
$6 \leq M < 7$	75 mi (121 km)	13
$7 \leq M < 8$	125 mi (201 km)	2
$M \geq 8$	200 mi (322 km)	0

NOTE: <https://earthquake.usgs.gov/earthquakes/search/> accessed on 16 May 2024.

Table 15.
Selected historical earthquakes with M4.5+ since 1 Jan 1800 (USGS).

Date	Name	M	Latitude (deg)	Longitude (deg)	Depth (km)	Distance (km)
18 Oct 1989	The 1989 Loma Prieta, California Earthquake	6.9	37.036	-121.880	17.2	96
18 Apr 1906	The 1906 San Francisco, California Earthquake	7.9	37.750	-122.550	11.7	14
25 Jun 1838	The 1838 San Andreas Fault, California Earthquake	7.4	37.30	-122.150	-	59

NOTE: <https://earthquake.usgs.gov/earthquakes/search/> accessed on 16 May 2024.

5.3. Seismic Hazard Curves

A local (site-specific) probabilistic seismic hazard analysis (PSHA) has not been performed for this project. The mean seismic hazard curve for the peak horizontal ground acceleration (PGA) and Site Class E was generated using the 2018 NSHM, as shown in Figure 39. The PGA corresponding to selected common values of return periods were interpolated from this mean hazard curve and are shown in Table 15.

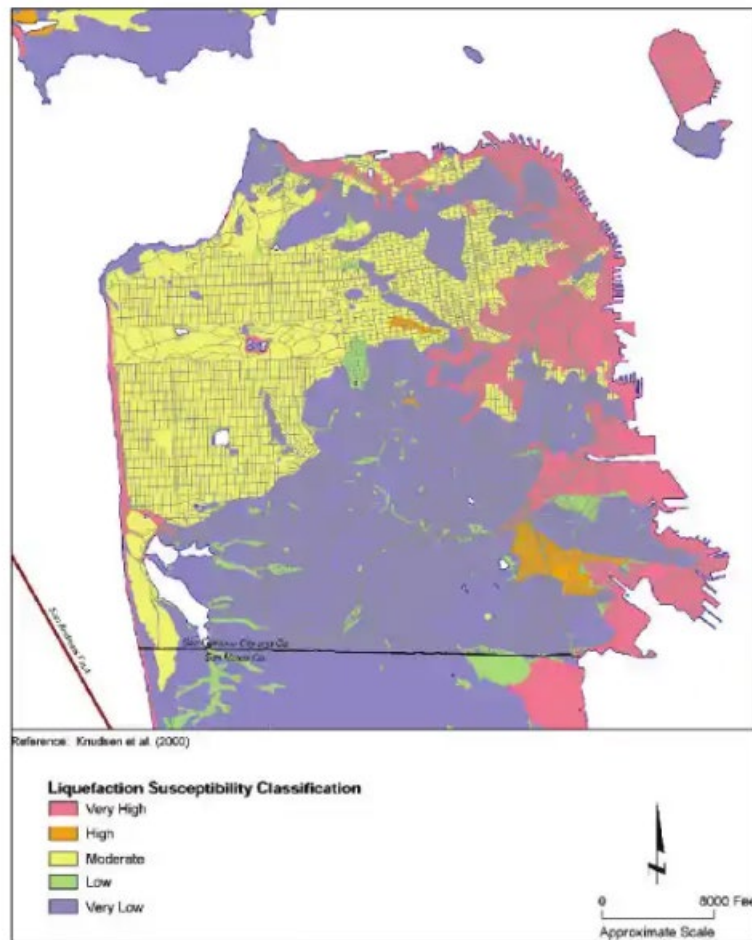


Figure 41. USGS NEHRP Site Classification for the San Francisco area

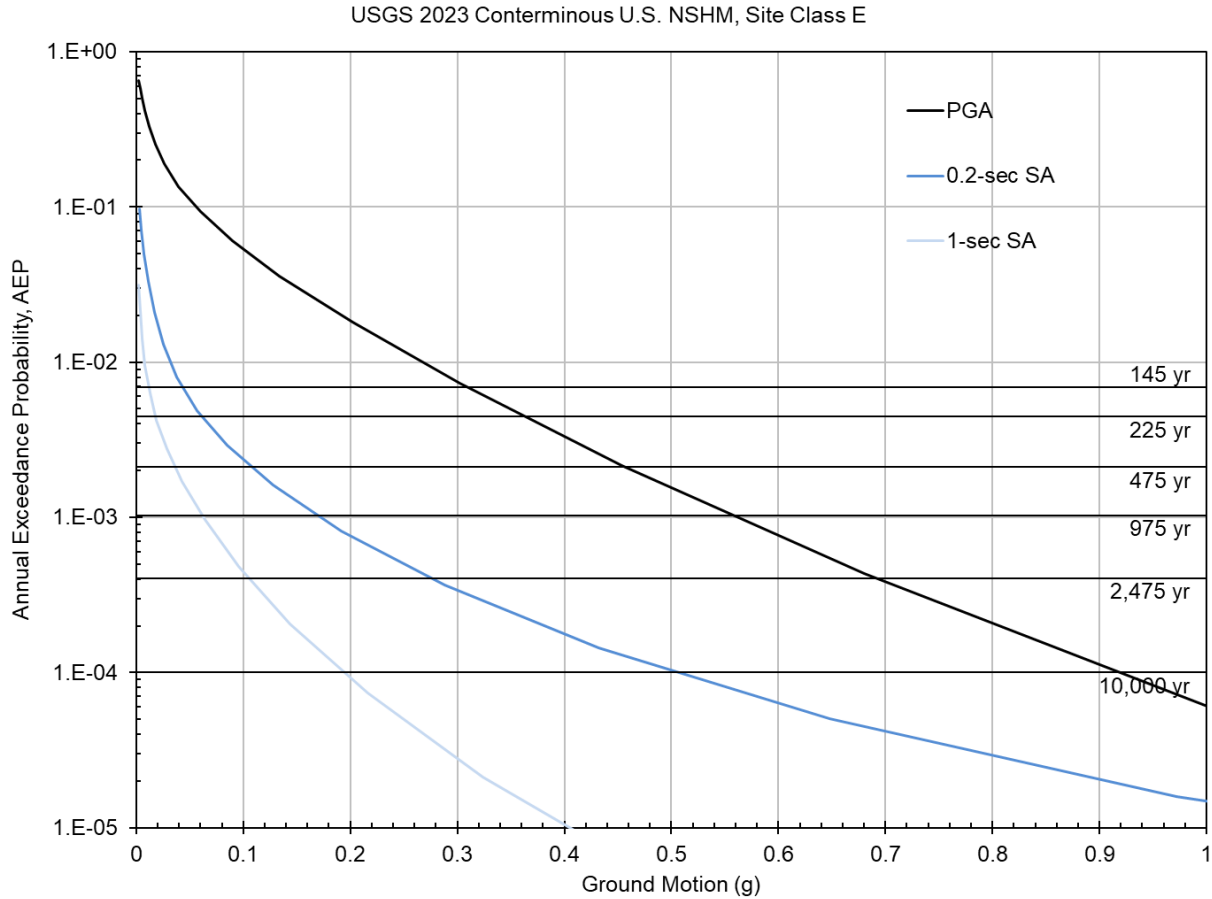


Figure 42. Seismic hazard curve for PGA and Site Class E (USGS 2018)

Table 16.

Peak horizontal ground acceleration summary for Site Class E (USGS 2018).

Return Period (years)	PGA (g)
145	0.309
225	0.359
475	0.457
975	0.551
2,475	0.690
10,000	0.909

For the purposes of this SQRA, a probabilistic return period and corresponding ground motion value were selected by the SQRA team that were judged appropriate for the potential failure mode and consequences, and the rationale was provided in Chapter 7 and Appendix A.

5.4. Disaggregation

The disaggregated seismic hazard for the PGA is shown in Figure 41 and Figure 42 using the USGS Earthquake Hazard Toolbox (<https://earthquake.usgs.gov/nshmp/>). Since shear wave velocity data was

not available for the site, Site Class E corresponding to an average v_{s30} of 150 m/s (492 ft/s) was used to perform the disaggregation and generate these figures.

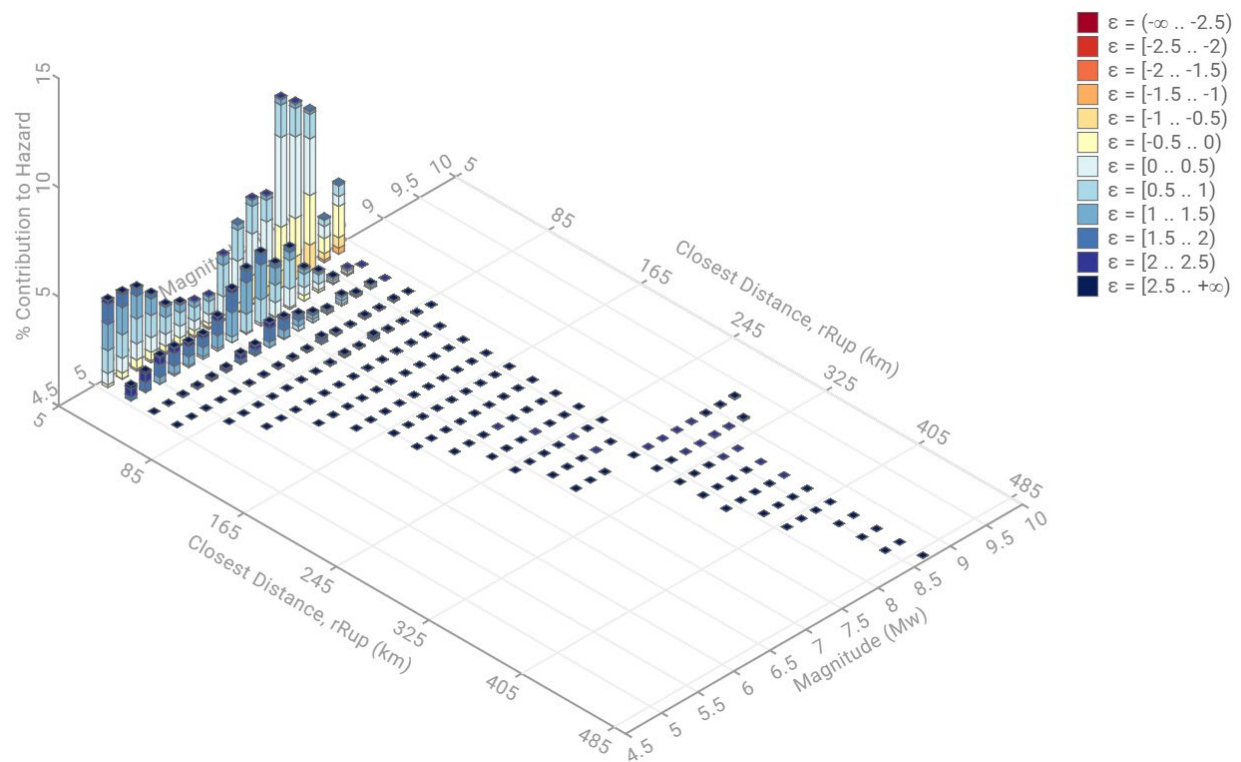


Figure 43. Probabilistic seismic hazard disaggregation for PGA and Site Class E with 1/ 145 AEP (USGS 2018)

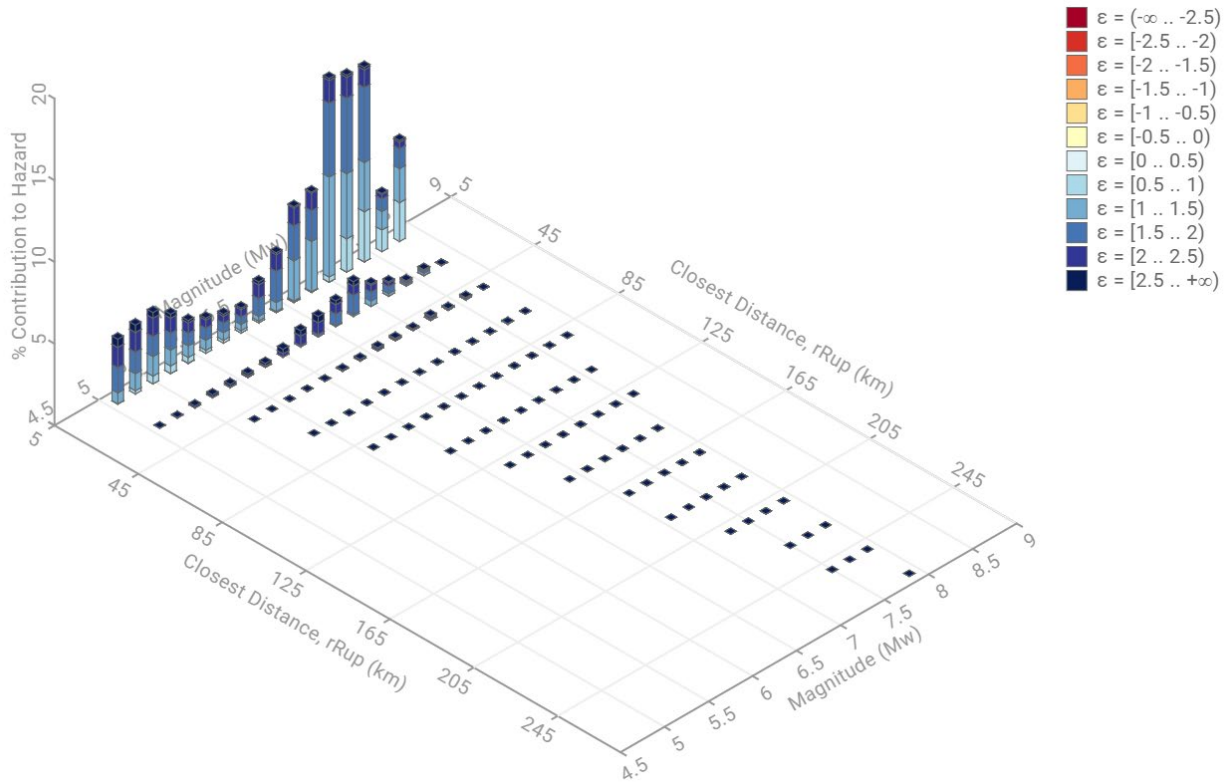


Figure 44. Probabilistic seismic hazard disaggregation for PGA and Site Class E with 1/ 2475 AEP (USGS 2018)

The disaggregation suggests that the primary contributors to the seismic hazard at the site include a M_w 8.0 earthquake on the San Andreas fault at a distance of less than 20 kilometers.

6. Consequences

6.1. Background

USACE has established a national standard of modeling procedures to support the estimation of consequences for breach and non-breach flood inundation scenarios for a range of loading conditions.

The San Francisco Waterfront Risk Assessment team developed baseline consequence estimates for breach and non-breach inundation scenarios, uncertainty statistics for life loss estimates, and inundation mapping products. The difference between breach and non-breach consequences for a particular loading condition is the incremental consequences (i.e., those directly attributable to the levee breach for that loading condition). This information was used by the risk assessment team to assign an appropriate order-of-magnitude estimate for the risk-driving potential failure modes.

6.2. Breach and Inundation Scenarios

For a detailed description of the breach scenarios and assumptions please see Section 4.6. Consequences were ran for the TSP for years 2090 and 2140 at various breach locations. In addition, Alternative G was ran as well. Figure 43 displays the breach locations for the TSP second action.

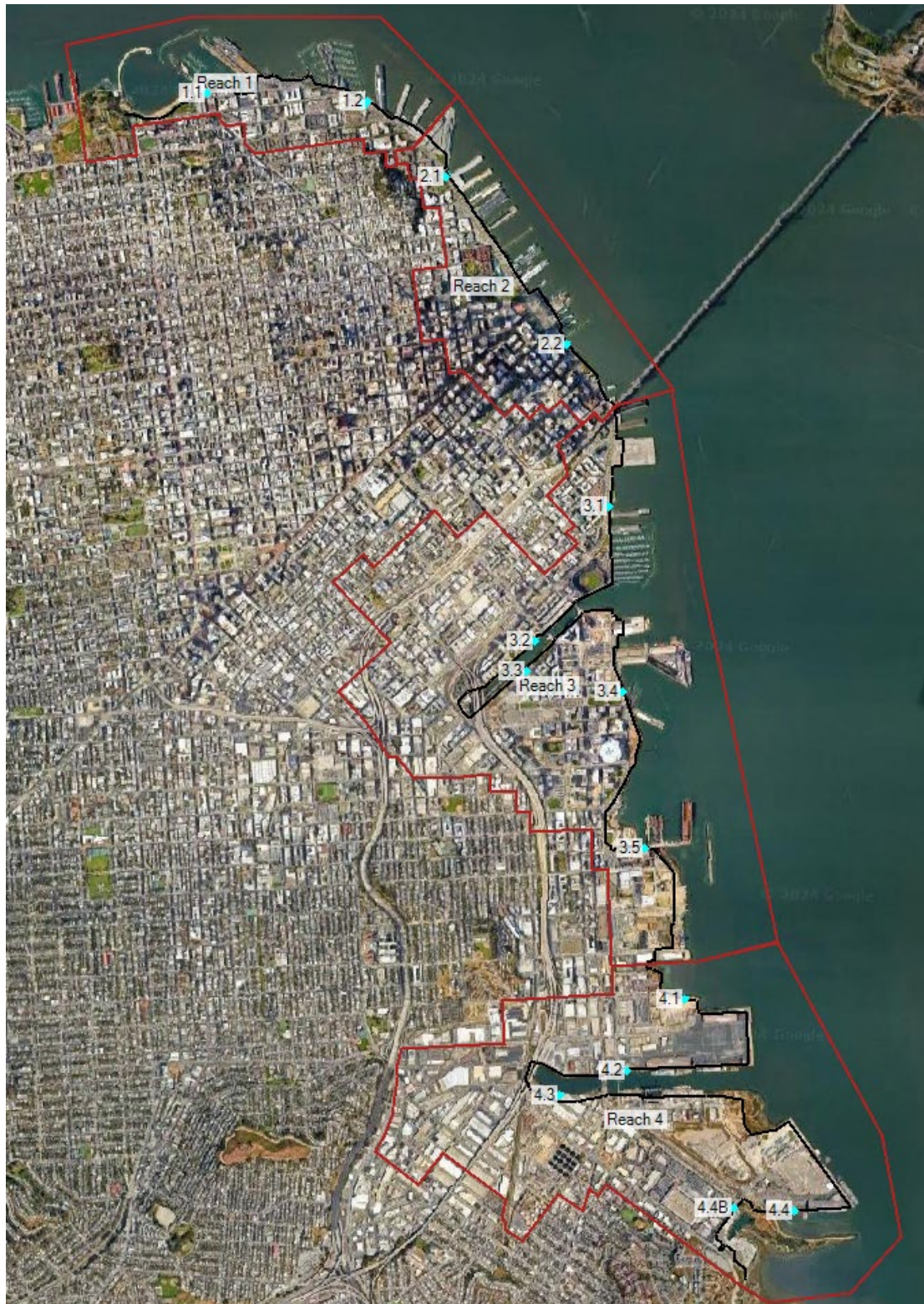


Figure 45. Second Action TSP Approximate Breach Locations

6.3. Description of Impacted Area

6.4. Life Loss Methodology

This consequence modeling was performed using USACE's life loss and direct property damage estimation software LifeSim version 2.1.3. To determine the percentage of population at risk (PAR) within a structure that is warned and mobilized over time, several parameters are used within LifeSim to estimate the probable values of warning and mobilization percentages at each time step. These include when hazards are identified and warnings will be issued (hazard identification and delays), how long they will take to become effective (warning diffusion), and the rate at which PAR will mobilize in response (mobilization or Protective Action Initiation (PAI)). Figure 44 is an example breach warning and response timeline.

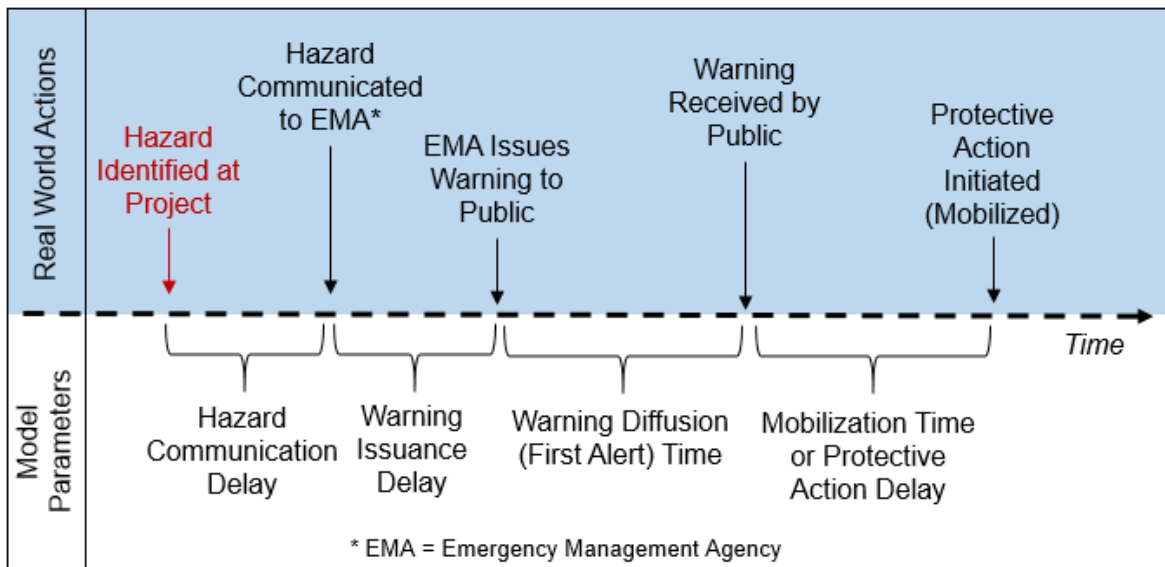


Figure 46. Warning and response timeline.

6.5. Population at Risk

Population at risk (PAR) is defined as the number of people downstream of a leveed area that would be subject to inundation during a flood hazard event. Estimates of PAR were generated using LifeSim for breach and non-breach inundation scenarios. The estimated number of inundated structures and PAR are summarized in Table 18 and Table 19.

Estimated number of inundated structures Second Action 2140

Breach Location	Breach	Non-Breach	Incremental
1.1	6,765	0	6,765
1.2	7,212	0	7,212
2.1	5,885	0	5,885
2.2	0	0	0

Breach Location	Breach	Non-Breach	Incremental
3.1	8,435	0	8,435
3.2	9,066	0	9,066
3.3	9,058	0	9,058
3.4	9,092	0	9,092
3.5	86	0	86
4.1	560	0	560
4.2	569	0	569
4.3	648	0	648
4.4	274	0	274

Table 20.
Estimated number of inundated structures Alternative G

Breach Location	Breach	Non-Breach	Incremental
1.1	3,810	0	3,810
1.2	4,107	0	4,107
2.1	3,071	0	3,071
2.2	3,821	0	3,821
3.1	4,860	0	4,860
3.2	4,088	0	4,088
3.3	220	0	220
4.1	131	0	131
4.2	421	0	421
4.3	363	0	363

Table 21, respectively.

Table 17.
Estimated number of inundated structures First Action 2090.

Breach Location	Breach	Non-Breach	Incremental
2.1	30	0	30
2.2	132	0	132

Breach Location	Breach	Non-Breach	Incremental
3.1	24	0	24
3.2	0	0	0
3.3	127	0	127
3.4	127	0	127
3.5	24	0	24
4.1	24	0	24
4.2	31	0	31
4.3	122	0	122
4.4	24	0	24

Table 18.**Estimated number of inundated structures Second Action 2140**

Breach Location	Breach	Non-Breach	Incremental
1.1	6,765	0	6,765
1.2	7,212	0	7,212
2.1	5,885	0	5,885
2.2	0	0	0
3.1	8,435	0	8,435
3.2	9,066	0	9,066
3.3	9,058	0	9,058
3.4	9,092	0	9,092
3.5	86	0	86
4.1	560	0	560
4.2	569	0	569
4.3	648	0	648
4.4	274	0	274

Table 19.**Estimated number of inundated structures Alternative G**

Breach Location	Breach	Non-Breach	Incremental
1.1	3,810	0	3,810
1.2	4,107	0	4,107
2.1	3,071	0	3,071
2.2	3,821	0	3,821

Breach Location	Breach	Non-Breach	Incremental
3.1	4,860	0	4,860
3.2	4,088	0	4,088
3.3	220	0	220
4.1	131	0	131
4.2	421	0	421
4.3	363	0	363

Table 20.
Estimated mean population at risk First Action 2090.

Breach Location	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
2.1	222	13	0	0	222	13
2.2	14,486	1,052	0	0	14,486	1,052
3.1	211	13	0	0	211	13
3.2	0	0	0	0	0	0
3.3	457	276	0	0	457	276
3.4	457	276	0	0	457	276
3.5	211	13	0	0	211	13
4.1	211	13	0	0	211	13
4.2	263	35	0	0	263	35
4.3	1,247	95	0	0	1,247	95
4.4	211	13	0	0	211	13

Table 21.
Estimated mean population at risk Second Action 2140.

Breach Location	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
1.1	131,724	23,593	0	0	131,724	23,593
1.2	139,671	25,554	0	0	139,671	25,554
2.1	122,112	21,080	0	0	122,112	21,080
2.2	0	0	0	0	0	0
3.1	156,200	29,956	0	0	156,200	29,956
3.2	156,983	30,966	0	0	156,983	30,966

Breach Location	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
3.3	156,201	30,829	0	0	156,201	30,829
3.4	157,728	31,051	0	0	157,728	31,051
3.5	814	86	0	0	814	86
4.1	6,078	570	0	0	6,078	570
4.2	6,335	587	0	0	6,335	587
4.3	7,444	683	0	0	7,444	683
4.4	3,004	249	0	0	3,004	249

Table 22.**Estimated mean population at risk Second Action Alternative G 2140.**

Breach Location	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
1.1	121,399	16,093	0	0	121,399	16,093
1.2	125,455	16,934	0	0	125,455	16,934
2.1	113,772	13,915	0	0	113,772	13,915
2.2	121,361	16,112	0	0	121,361	16,112
3.1	81,832	16,645	0	0	81,832	16,645
3.2	47,960	11,673	0	0	47,960	11,673
3.3	2,985	1,043	0	0	2,985	1,043
4.1	1,272	165	0	0	1,272	165
4.2	4,479	385	0	0	4,479	385
4.3	3,545	318	0	0	3,545	318

6.6. Life Loss Model Inputs and Parameters

6.6.1. Structure Inventory

The structure inventory was developed using the USACE National Structure Inventory (NSI) version 2022 as the base dataset. The NSI is developed from estimated information regarding the locations, building types, population, values, and other relevant information for all residential, commercial, industrial, agricultural, public, and private structures across the nation. Data sources for this information include national level parcel data, Census data, building footprints, and other sources for employment and school information. The value of structures, contents, and vehicles were indexed to 2024 price levels. Additional calibration on the structure point placement was accomplished using aerial imagery and inundation maps for both high and low flow scenarios.

6.6.2. Road Network and Destination Points

Evacuation simulation was not performed.

6.6.3. Emergency Preparedness

Because of uncertainty associated with emergency operations and response during a flood event, it was necessary to account for a wide range of possible warning and evacuation outcomes. A series of preset curves are included in LifeSim. These curves were developed using the research from Mileti and Sorensen (2014). For this study, the “Preparedness Unknown” curves were chosen for the warning parameters (Warning Issuance Delay and Warning Diffusion). The Emergency Preparedness parameters are described in the sections below.

6.6.4. Relative Hazard Identification

The Hazard Identification time is the time at which a hazard is identified (i.e., breach) relative to when it occurs (the actual breach initiation time). Two different warning scenarios were simulated with different ranges of hazard identification time: minimal warning and ample warning.

For overtopping scenarios, warning is relative to the time that overtopping begins, not breach time. The Relative Hazard Identification Times used for this study are summarized below.

Table 23.
Relative hazard identification times.

Warning Scenario	Distribution Type	Minimum (hr)	Maximum (hr)
Minimal	Uniform	-3	0.5
Ample	None	-24	-24

6.6.5. Hazard Communication Delay

The Hazard Communication Delay is the time that it would take from when the hazard is identified to when the emergency planning zone (EPZ) representatives would be notified. For example, if a breach occurs when no one is observing the project then the emergency managers could be notified after the hazard is identified. The hazard communication delay ranged between 0.01 and 0.5 hour and assumed a uniform distribution.

6.6.6. Warning Issuance Delay

The Warning Issuance Delay is the time it takes from when the emergency managers receive the notification of the imminent hazard to when they issue the first evacuation order to the public. Although the range of possible warning issuance delay is randomly sampled from 0 and 6 hours (300 minutes) after officials are notified of the flood hazard using a Lindell distribution, it is positively skewed so that results from 0 to 1.5 hours (90 minutes) are more likely.

6.6.7. Warning Diffusion

The preset “Preparedness Unknown” warning diffusion curves were used. The curves utilize a uniform distribution, and the warning diffusion curves are sampled during each Monte Carlo iteration in LifeSim.

The upper bound of the curve reaches 100% diffusion just after 1.5 hours (100 minutes), and the lower bound reaches 100% diffusion after 6 hours (360 minutes).

6.6.8. Protective Action Initiation

Protective Action Initiation (PAI) is the rate at which PAR takes action after receiving an evacuation order (warning). Unlike the warning diffusion curves, the PAI “Preparedness Unknown” curve includes a perception element as well. The perception element describes a PAR as being aware of their flood risk (Perception = Likely to Impact) or generally unaware that they are at risk of being flooded (Perception = Unlikely to Impact). Figure 45 displays the “Preparedness Unknown, Perception Unknown” curve was used for all areas. Figure 46 displays the coastal PAI curve that was used as a sensitivity comparison and for the overtopping scenarios.

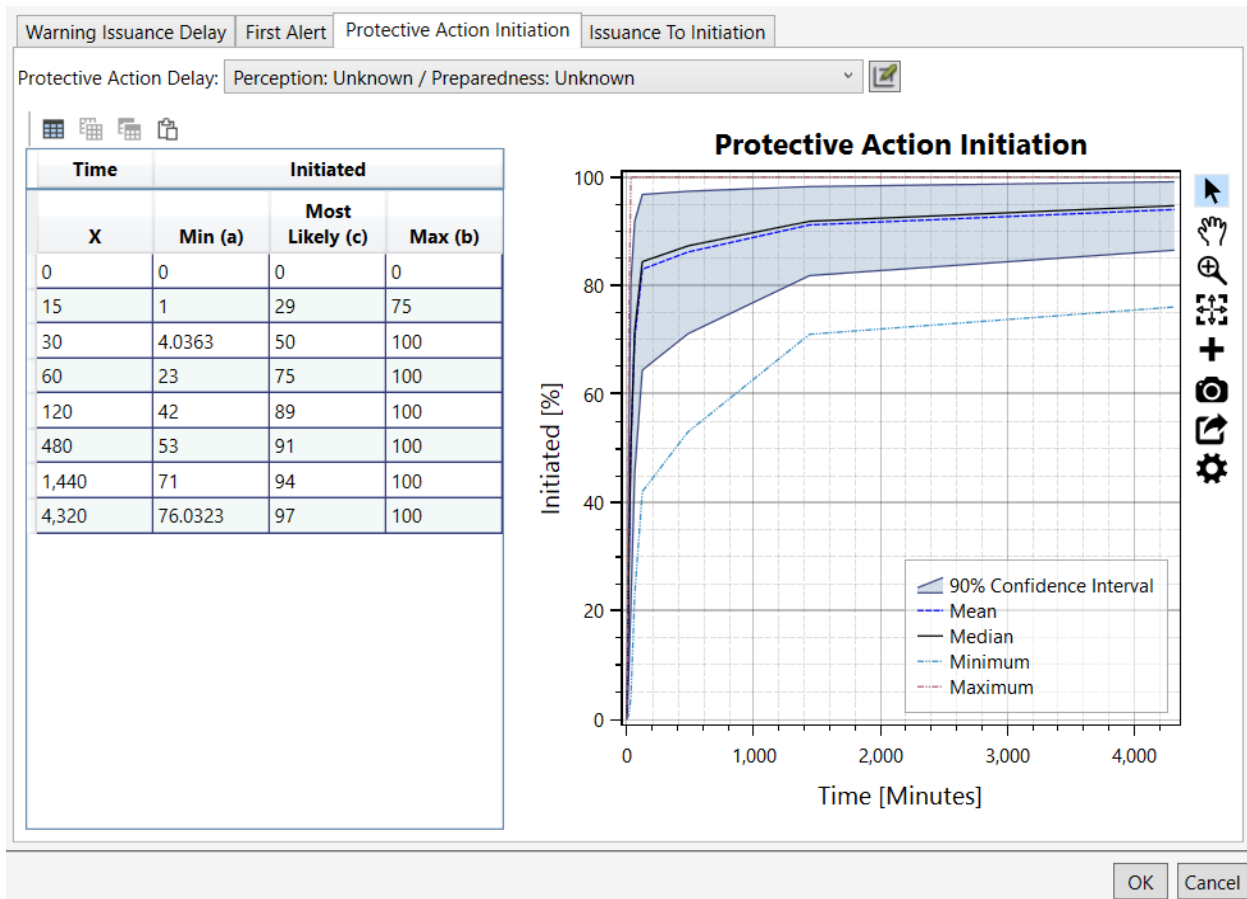


Figure 47. PAI curve for preparedness unknown, perception unknown.

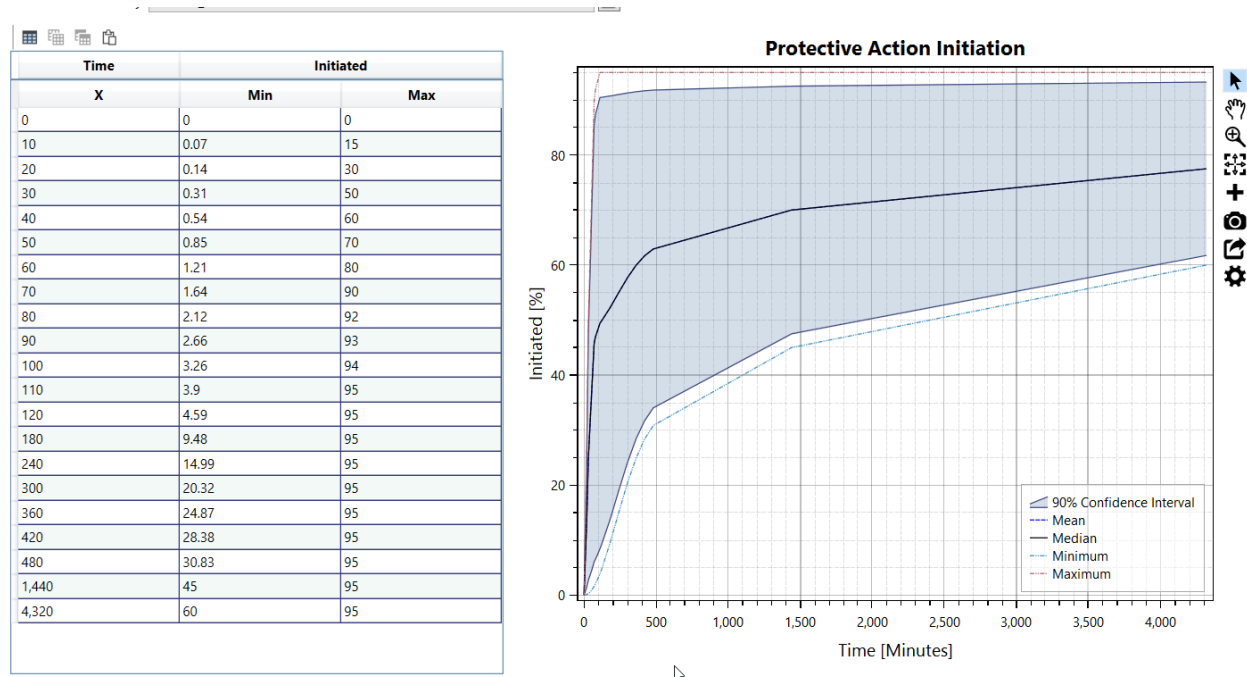


Figure 48. Coastal PAI curve for preparedness unknown, perception unknown.

6.7. Direct Life Loss

Estimates of direct life loss were generated using LifeSim for breach and non-breach inundation scenarios. LifeSim utilizes Monte Carlo uncertainty analysis, and 1,000 iterations were performed for each scenario. The estimated direct life loss by breach location and loading scenario are summarized in Table 25 for minimal warning and **Error! Reference source not found.** for ample warning. While the direct breach and non-breach results represent the outcome from model simulations, the direct incremental life loss results were obtained by subtracting the breach and non-breach results.

Table 24.

Estimated Incremental direct life loss for First Action 2090 Minimal Warning.

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_2.1_SynTide_1Per_IC_2090	0	0
Breach_2.2_SynTide_1Per_IC_2090	0	0
Breach_3.1_SynTide_1Per_IC_2090	0	0
Breach_3.2_SynTide_1Per_IC_2090	0	0
Breach_3.3_SynTide_1Per_IC_2090	0	0

Breach_3.4_SynTide_1Per_IC_2090	0	0
Breach_3.5_SynTide_1Per_IC_2090	0	0
Breach_4.1_SynTide_1Per_IC_2090	0	0
Breach_4.2_SynTide_1Per_IC_2090	0	0
Breach_4.3_SynTide_1Per_IC_2090	0	0
Breach_4.4_SynTide_1Per_IC_2090	0	0

Table 25.**Estimated Incremental direct life loss for First Action 2090 Ample Warning.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_2.1_SynTide_1Per_IC_2090	0	0
Breach_2.2_SynTide_1Per_IC_2090	0	0
Breach_3.1_SynTide_1Per_IC_2090	0	0
Breach_3.2_SynTide_1Per_IC_2090	0	0
Breach_3.3_SynTide_1Per_IC_2090	0	0
Breach_3.4_SynTide_1Per_IC_2090	0	0
Breach_3.5_SynTide_1Per_IC_2090	0	0
Breach_4.1_SynTide_1Per_IC_2090	0	0
Breach_4.2_SynTide_1Per_IC_2090	0	0
Breach_4.3_SynTide_1Per_IC_2090	0	0
Breach_4.4_SynTide_1Per_IC_2090	0	0

Table 26.**Estimated Incremental direct life loss for Second Action 2140 Minimal Warning.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_1.1_SynTide_1Per_HC_2140	16	1
Breach_1.2_SynTide_1Per_HC_2140	31	3
Breach_2.1_SynTide_1Per_HC_2140	12	1
Breach_2.2_SynTide_1Per_HC_2140	0	0
Breach_3.1_SynTide_1Per_HC_2140	180	59
Breach_3.2_SynTide_1Per_HC_2140	180	69
Breach_3.3_SynTide_1Per_HC_2140	208	62
Breach_3.4_SynTide_1Per_HC_2140	231	70
Breach_3.5_SynTide_1Per_HC_2140	11	1
Breach_4.1_SynTide_1Per_HC_2140	28	5
Breach_4.2_SynTide_1Per_HC_2140	32	6
Breach_4.3_SynTide_1Per_HC_2140	109	8
Breach_4.4_SynTide_1Per_HC_2140	11	1

Table 27.**Estimated Incremental direct life loss for Second Action 2140 Ample Warning.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_1.1_SynTide_1Per_HC_2140	4	0
Breach_1.2_SynTide_1Per_HC_2140	8	1
Breach_2.1_SynTide_1Per_HC_2140	2	0
Breach_2.2_SynTide_1Per_HC_2140	0	0
Breach_3.1_SynTide_1Per_HC_2140	36	9
Breach_3.2_SynTide_1Per_HC_2140	28	9
Breach_3.3_SynTide_1Per_HC_2140	35	10
Breach_3.4_SynTide_1Per_HC_2140	37	11
Breach_3.5_SynTide_1Per_HC_2140	1	0
Breach_4.1_SynTide_1Per_HC_2140	5	1
Breach_4.2_SynTide_1Per_HC_2140	5	1
Breach_4.3_SynTide_1Per_HC_2140	13	1
Breach_4.4_SynTide_1Per_HC_2140	2	0

Table 28.**Estimated Incremental direct life loss for Second Action 2140 Alternative G Minimal Warning.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_G1.1_SynTide_1Per_HC_G_2140	21	2
Breach_G1.2_SynTide_1Per_HC_G_2140	47	6
Breach_G2.1_SynTide_1Per_HC_G_2140	5	0
Breach_G2.2_SynTide_1Per_HC_G_2140	16	2
Breach_G3.1_SynTide_1Per_HC_G_2140	187	77
Breach_G3.2_SynTide_1Per_HC_G_2140	54	19
Breach_G3.3_SynTide_1Per_HC_G_2140	4	0
Breach_G4.1_SynTide_1Per_HC_G_2140	4	0
Breach_G4.2_SynTide_1Per_HC_G_2140	15	1
Breach_G4.3_SynTide_1Per_HC_G_2140	6	0

Table 29.**Estimated Incremental direct life loss for Second Action 2140 Alternative G Ample Warning.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_G1.1_SynTide_1Per_HC_G_2140	6	1
Breach_G1.2_SynTide_1Per_HC_G_2140	12	2
Breach_G2.1_SynTide_1Per_HC_G_2140	1	0
Breach_G2.2_SynTide_1Per_HC_G_2140	5	1
Breach_G3.1_SynTide_1Per_HC_G_2140	23	9
Breach_G3.2_SynTide_1Per_HC_G_2140	12	4
Breach_G3.3_SynTide_1Per_HC_G_2140	1	0
Breach_G4.1_SynTide_1Per_HC_G_2140	1	0
Breach_G4.2_SynTide_1Per_HC_G_2140	2	0
Breach_G4.3_SynTide_1Per_HC_G_2140	1	0

Table 30.**Estimated Incremental direct life loss for Second Action 2140 Alternative G Minimal Warning Coastal PAI Curve.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_G1.1_SynTide_1Per_HC_G_2140	34	4
Breach_G1.2_SynTide_1Per_HC_G_2140	76	12
Breach_G2.1_SynTide_1Per_HC_G_2140	8	1
Breach_G2.2_SynTide_1Per_HC_G_2140	30	4
Breach_G3.1_SynTide_1Per_HC_G_2140	212	85
Breach_G3.2_SynTide_1Per_HC_G_2140	82	29
Breach_G3.3_SynTide_1Per_HC_G_2140	4	0
Breach_G4.1_SynTide_1Per_HC_G_2140	4	0
Breach_G4.2_SynTide_1Per_HC_G_2140	17	1
Breach_G4.3_SynTide_1Per_HC_G_2140	7	0

Table 31.**Estimated Incremental direct life loss for Second Action 2140 Alternative G Ample Warning Coastal PAI Curve.**

Breach Location	Day Mean Life Loss	Night Mean Life Loss
Breach_G1.1_SynTide_1Per_HC_G_2140	19	3
Breach_G1.2_SynTide_1Per_HC_G_2140	40	8
Breach_G2.1_SynTide_1Per_HC_G_2140	4	0
Breach_G2.2_SynTide_1Per_HC_G_2140	18	3
Breach_G3.1_SynTide_1Per_HC_G_2140	75	30
Breach_G3.2_SynTide_1Per_HC_G_2140	39	14
Breach_G3.3_SynTide_1Per_HC_G_2140	2	0

Breach_G4.1_SynTide_1Per_HC_G_2140	1	0
Breach_G4.2_SynTide_1Per_HC_G_2140	7	0
Breach_G4.3_SynTide_1Per_HC_G_2140	3	0

Table 32.

Estimated Incremental direct life loss for First Action 2090 Minimal Warning Coastal PAI Curve 1ft Overtopping.

Breach Location	Day Mean Life Loss	Night Mean Life Loss
3.1	0	0
3.2	0	0
3.3	0	0

Table 33.

Estimated Incremental direct life loss for First Action 2090 Ample Warning Coastal PAI Curve 1ft Overtopping.

Breach Location	Day Mean Life Loss	Night Mean Life Loss
3.1	0	0
3.2	0	0
3.3	0	0

Table 34.

Estimated Incremental direct life loss for First Action 2140 Minimal Warning Coastal PAI Curve 1ft Overtopping.

Breach Location	Day Mean Life Loss	Night Mean Life Loss
3.1	0	0
3.2	0	0
3.3	0	0

Table 35.

Estimated Incremental direct life loss for First Action 2140 Ample Warning Coastal PAI Curve 1ft Overtopping.

Breach Location	Day Mean Life Loss	Night Mean Life Loss
3.1	0	0
3.2	0	0
3.3	0	0

6.8. Economic Losses

Economic considerations help inform risk management decisions. Direct economic losses considered in this SQRA include direct property damages.

6.8.1. Direct Property Damage

An inventory of downstream property directly damaged by the inundation was developed using the structure inventory described in Section 6.6.1 which contains information for residential, commercial, industrial, agricultural, public and private structures as well as data for essential facilities such as hospitals, clinics, fire stations, police stations, emergency operation centers, schools and universities.

Direct property damage including structure, content, and vehicle damage was estimated in LifeSim using standard depth-percent damage curves specific to each structure occupancy type and structure stability curves which evaluate damage caused by combined depth and velocity.

The estimated mean direct property damage is summarized in Table 37.

Table 36.
Estimated mean direct property damage First Action 2090.

Breach Location	Breach	Non-Breach	Incremental
Breach_2.1_SynTide_1Per_IC_2090	\$27,000,000	\$0	\$27,000,000
Breach_2.2_SynTide_1Per_IC_2090	\$27,300,000	\$0	\$27,300,000
Breach_3.1_SynTide_1Per_IC_2090	\$17,300,000	\$0	\$17,300,000
Breach_3.2_SynTide_1Per_IC_2090	\$0	\$0	\$0
Breach_3.3_SynTide_1Per_IC_2090	\$20,000,000	\$0	\$20,000,000
Breach_3.4_SynTide_1Per_IC_2090	\$20,000,000	\$0	\$20,000,000
Breach_3.5_SynTide_1Per_IC_2090	\$17,300,000	\$0	\$17,300,000
Breach_4.1_SynTide_1Per_IC_2090	\$17,300,000	\$0	\$17,300,000
Breach_4.2_SynTide_1Per_IC_2090	\$17,800,000	\$0	\$17,800,000
Breach_4.3_SynTide_1Per_IC_2090	\$47,700,000	\$0	\$47,700,000
Breach_4.4_SynTide_1Per_IC_2090	\$17,300,000	\$0	\$17,300,000

NOTE: FY 2024 price level

Table 37.
Estimated mean direct property damage Second Action 2140.

Breach Location	Breach	Non-Breach	Incremental
Breach_1.1_SynTide_1Per_HC_2140	\$3,892,600,000	\$0	\$3,892,600,000
Breach_1.2_SynTide_1Per_HC_2140	\$4,578,400,000	\$0	\$4,578,400,000
Breach_2.1_SynTide_1Per_HC_2140	\$3,048,100,000	\$0	\$3,048,100,000
Breach_2.2_SynTide_1Per_HC_2140	\$0	\$0	\$0
Breach_3.1_SynTide_1Per_HC_2140	\$6,638,700,000	\$0	\$6,638,700,000
Breach_3.2_SynTide_1Per_HC_2140	\$6,315,000,000	\$0	\$6,315,000,000
Breach_3.3_SynTide_1Per_HC_2140	\$6,258,300,000	\$0	\$6,258,300,000
Breach_3.4_SynTide_1Per_HC_2140	\$6,374,000,000	\$0	\$6,374,000,000
Breach_3.5_SynTide_1Per_HC_2140	\$225,500,000	\$0	\$225,500,000
Breach_4.1_SynTide_1Per_HC_2140	\$657,200,000	\$0	\$657,200,000
Breach_4.2_SynTide_1Per_HC_2140	\$662,600,000	\$0	\$662,600,000
Breach_4.3_SynTide_1Per_HC_2140	\$921,800,000	\$0	\$921,800,000
Breach_4.4_SynTide_1Per_HC_2140	\$336,200,000	\$0	\$336,200,000

NOTE: FY 2024 price level

Table 38.
Estimated mean direct property damage Second Action 2140 Alternative G.

Breach Location	Breach	Non-Breach	Incremental
Breach_G1.1_SynTide_1Per_HC_G_2140	\$3,185,700,000	\$0	\$3,185,700,000
Breach_G1.2_SynTide_1Per_HC_G_2140	\$3,547,800,000	\$0	\$3,547,800,000
Breach_G2.1_SynTide_1Per_HC_G_2140	\$2,560,600,000	\$0	\$2,560,600,000
Breach_G2.2_SynTide_1Per_HC_G_2140	\$3,166,800,000	\$0	\$3,166,800,000
Breach_G3.1_SynTide_1Per_HC_G_2140	\$2,335,900,000	\$0	\$2,335,900,000
Breach_G3.2_SynTide_1Per_HC_G_2140	\$1,675,900,000	\$0	\$1,675,900,000
Breach_G3.3_SynTide_1Per_HC_G_2140	\$202,500,000	\$0	\$202,500,000
Breach_G4.1_SynTide_1Per_HC_G_2140	\$186,900,000	\$0	\$186,900,000
Breach_G4.2_SynTide_1Per_HC_G_2140	\$522,200,000	\$0	\$522,200,000
Breach_G4.3_SynTide_1Per_HC_G_2140	\$358,200,000	\$0	\$358,200,000

NOTE: FY 2024 price level

7. Risk Assessment

7.1. Potential Failure Mode Analysis

A potential failure mode (PFM) is a unique set of conditions and/or sequence of events that could result in failure. For dam and levee safety risk assessments, failure generally results in breach or the formation of a gap in the dam or levee system through which water may flow uncontrolled onto the downstream flood plain or adjacent leveed area. It is characterized by the “sudden, rapid, and uncontrolled release of impounded water” as defined in FEMA P-93 (FEMA 2004), and FEMA P-1025 (FEMA 2015) expanded the definition of failure to include “liquid-borne solids” like mine tailings and dredge spoils.

A potential failure mode analysis (PFMA) is a facilitated process of identifying and fully describing PFMs. It is based on a team’s understanding of the project’s vulnerabilities from a review of existing data and conditions. The facilitated PFMA was performed on 02-04 April 2024. The PFMA team and their roles are provided in Table 40.

Table 39.
PFMA team.

Name	Organization/Office	Discipline/Role
Alexander Nelson, P.E.	USACE, Levee Safety Center	Hydraulic Engineer (Facilitator)
Axel Montalvo Bartolomei, P.E.	USACE, Levee Safety Center	Geotechnical Engineer (Co-facilitator)
BJ Evans, P.E.	USACE, Risk Management Center	Geotechnical Engineer
Jamie Evans, P.E.	USACE, Memphis District	Engineering Geologist
Tacy Jensen	USACE, Tulsa District	Project Manager
Glenn Fulton	USACE, Tulsa District	Consequence Specialist
Samantha Palmason	USACE, Tulsa District	Hydraulic Engineer
Jackson Pruss	USACE, Tulsa District	Geographer (Note taker)
Matthew Wickens	Port of San Francisco	Project Manager
Steven Reel	Port of San Francisco	Project Manager
Chris Horiuchi	Port of San Francisco	Project Manager

Advance preparation includes collecting and reviewing the background and performance data, site characterization, hydrologic and seismic hazards, hydraulic and consequence modeling, and performing any screening or limited supporting analysis. Prior to the PFMA, a site visit was performed to help the team looking for clues as to how the dam or levee system and appurtenant structures may be vulnerable to failure.

A site visit and review of background data was initially conducted followed by brainstorming of PFMs by the SQRA team. Table 41 provides the list of PFMs brainstormed by the SQRA team. PFMs related to sabotage or terrorism were beyond the scope of this risk assessment. From the list of brainstormed PFMs, the SQRA team identified the PFMs judged to be risk drivers. PFMs that were judged to be non-risk drivers were excluded from further consideration, and the rationale is provided in Appendix A.

Table 40.
Potential failure modes.

Potential Failure Mode	Description
Levee embankments	
PFM 1	Embankment overtopping erosion breach
PFM 2	Backwards erosion piping (BEP) through the foundation
PFM 3	Concentrated leak erosion through a crack in the embankment
PFM 4	CLE along a pipe penetration in the levee or foundation
PFM 5	CLE along a tie-in of a levee (with floodwall, with high ground, etc.)
PFM 6	Hydraulic fracture in foundation leads to CLE
PFM 7	Seismic event leads to levee crest deformation due to lateral spreading and/or liquefaction
PFM 8	Settlement of the levee leads to more frequent overtopping
PFM 9	Slope instability leads to loss of levee crest
PFM 10	Wave erosion progresses to loss of crest
PFM 11	Wave overtopping erosion of the embankment
PFM 12	Global instability of the levee embankment and foundation
PFM 13	Animal burrows leads to shortened seepage path for BEP or CLE
PFM 14	Floatation of lightweight material in tie-in (high ground) leads to seepage and stability mechanisms that results in flanking of the levee
PFM 15	CLE along vegetation roots beneath the levee
PFM 16	Waterside erosion of the levee embankment leads to loss of crest
PFM 17	Wake or prop wash scour leads to levee breach
PFM 18	Internal erosion out of a pressurized pipe
PFM 19	Internal migration into parallel storm sewer
PFM 20	Scour from storm sewer discharge erodes the levee
PFM 21	Tsunami overtops and breaches the levee
Bulkhead walls / Wharf walls / Retaining walls	
PFM 22	Contact erosion along foundation interface with rock dike
PFM 23	Suffusion
PFM 24	Structural, stability, or overtopping failure of pier floodwalls
PFM 25	Failure of a new wharf structure fails the bulkhead
PFM 26	Failure of an existing wharf structure fails the bulkhead
PFM 27	Internal migration into parallel storm sewer behind the wall
PFM 28	Global instability of bulkhead wall
PFM 29	Sliding, bearing, overturning failure of bulkhead wall
PFM 30	Prop wash erosion undermines bulkhead wall
PFM 31	Seismic event leads to a global instability of the wall and soil
PFM 32	Structural failure of bulkhead wall reinforcement
PFM 33	Ship impact failure of the bulkhead wall
PFM 34	Surcharge loading fails the bulkhead wall
PFM 35	Tree within VFZ topples and damages wall
PFM 36	Overtopping erosion behind the bulkhead wall

Potential Failure Mode	Description
PFM 37	Wave impact failure of bulkhead wall
PFM 38	CLE along penetrations through bulkhead wall
PFM 39	Collapse of BART tunnel (seismic, structural, etc.) fails bulkhead wall
PFM 40	Construction of planned BART tunnel leads to failure of bulkhead wall
PFM 41	Settlement of the bulkhead wall and fill leads to more frequent overtopping
PFM 42	Seismic failure of external infrastructure and buildings
PFM 43	Failure of the wharf leads to bulkhead wall failure
PFM 44	Failure of dewatering measures leads to construction personnel risk
PFM 45	Overdredging leads to global stability of bulkhead wall
PFM 46	Fire at a pier compromises bulkhead wall
PFM 47	Settlement-induced bending moment failure
Surface walls (T-walls, I-walls, L-walls, etc.)	
PFM 48	Overtopping erosion behind surface floodwall
PFM 49	Sliding, bearing, overturning failure of surface floodwall
PFM 50	Ship impact failure of surface floodwall
PFM 51	Wave impact failure of surface floodwall
PFM 52	Seismic event leads to global stability or structural failure of floodwall
PFM 53	BEP through foundation under surface floodwall
PFM 54	Unauthorized modification to surface floodwall leads to ineffectiveness
PFM 55	Vehicle impact to surface floodwall
PFM 56	Structural failure of surface floodwall reinforcement
PFM 57	CLE along interface between surface floodwall and soil
PFM 58	Animal burrows leads to shortened seepage path for BEP or CLE under a surface floodwall
PFM 59	Fire at a pier compromises surface floodwall
Closures and deployables	
PFM 60	Operational failure to make a closure
PFM 61	Train impact failure closure
PFM 62	Vehicle impact failure of closure
PFM 63	Structural failure of a closure
PFM 64	Leakage leads to erosion which fails the adjacent wall
PFM 65	Seismic event occurs and fails closure sill
PFM 66	Debris impact failure of closure
Other (2nd Action and EWN)	
PFM 67	Pier walls around wharves, which become line of protection after second action, fail (structural, stability, overtopping)
PFM 68	Settlement and/or wave erosion of coarse beach EWN leads to ineffective wave reduction and wave overtopping breach occurs
PFM 69	Debris blockage of flap gate leads to backflow
PFM 70	Failure of San Francisco Transition Structure (SFTS) leads to backflow through tunnel and flooding of interior (due to ship impact, seismic, etc.)

7.2. Risk Assessment

A semi-quantitative risk assessment (SQRA) is a facilitated process of making order-of-magnitude estimates of likelihood of failure and average incremental consequences. It is based on limited numerical data and judgment. The facilitated SQRA was performed using the team listed in Table 40.

Risk drivers from the PFMA were fully developed and semi-quantitatively evaluated. This includes a complete description of the failure mechanism from initiation to failure, nature of the failure (e.g., breach characteristics) and consequences, event tree, pertinent background and performance data, likelihood factors, estimated likelihood of failure and associated consequences, intervention, plotting on a risk matrix, and any sources of significant uncertainty in the estimates. Lastly, any recommendations for risk-reduction actions, instrumentation and monitoring, additional data, or analysis were discussed. Table 42 summarizes the PFMs judged to be risk drivers.

Table 41.
Risk-driving potential failure modes.

Potential Failure Mode	Description
PFM 01	Levee embankment overtopping breach

The incremental risk (due to failure) considers likelihood of failure and incremental consequences. The likelihood of failure is a function of both the likelihood of the loading condition that could lead to failure (i.e., critical loading) and the likelihood of failure given the loading condition (i.e., conditional probability of failure or system response probability). During the SQRA, order-of-magnitude estimates were made for both likelihood of failure and incremental consequences (based on estimated consequences provided by the MMC Production Center, private and government sources, and the team's judgment) for each risk-driving potential failures mode. The evaluation of each risk-driving PFM was documented as well as any sources of significant uncertainty.

The risk assessment discussed all future planning horizons and future project adaptability. However, the risk estimates are made only for the expected condition after the first action at the end of the project design life of 50 years – the year 2090, considering the range of sea level rise conditions.

7.3. PFM 01: Levee Embankment Overtopping Breach

7.3.1. Description

Water levels, due to a combination of high tides and a storm event, are forecasted to exceed the levee crest elevation. Intervention efforts to raise the levee are either not pursued or unsuccessful due to time and resource constraints. As the water overtops the levee, the grass slope protection fails, and bare soil is exposed. Overtopping flows initiate erosion of the soil, exposing a knick point to start a headcut. There are no effective, non-erodible features that would arrest the erosion. The headcut erosion progresses through the embankment to the waterside edge of the levee crown, resulting in increased flow, downcutting, and widening of a breach.

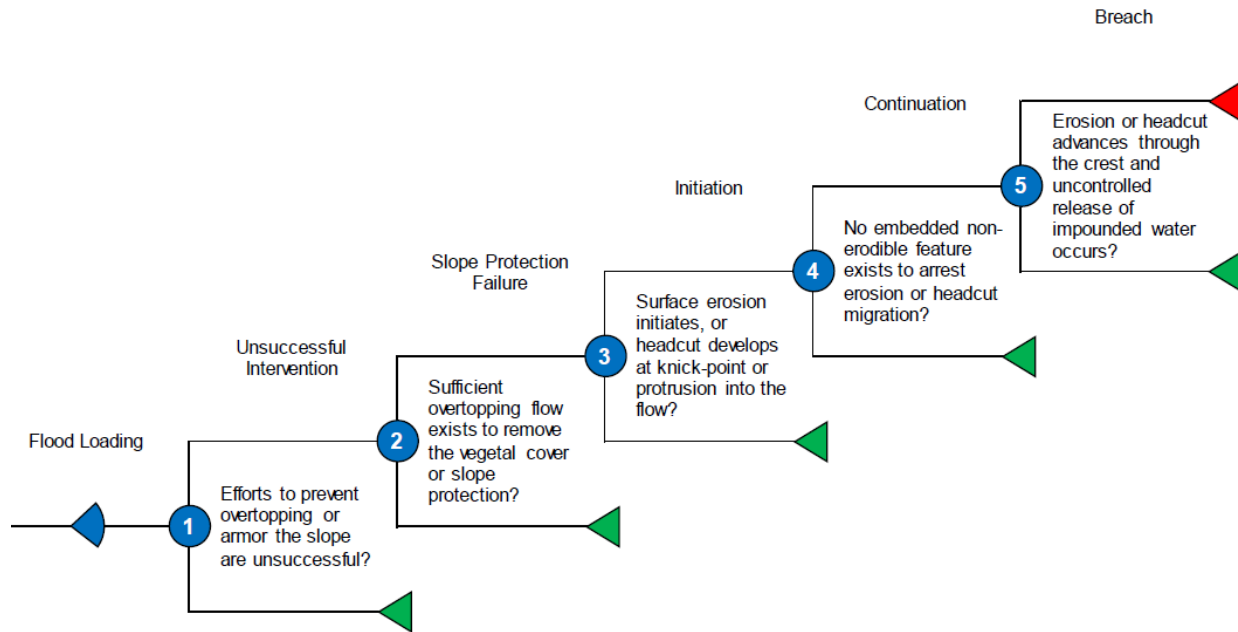


Figure 49. Event tree for PFM 1.

7.3.2. Background

The typical levee section is expected to have a 10-foot crown width and 1V:3H side slopes on the landside. Some areas, due to proximity to roadways, parking lots, and other features, may have flattened slopes. Good grass cover should be achievable in this climate, with adequate rainfall and minimal excessive heat or drought. Many parks adjacent to the alignment have continual grass cover in good condition. The levee will be constructed of a lean clay (CL and some CH), which is less erodible material.

In some areas, rather than constructing a traditional levee, the plan is to adopt an ecotone levee section. An ecotone levee has much flatter slopes of 1V:10H to 1V:30H, a much wider embankment footprint, and additional wetland vegetation of native grasses and similar plants. The purpose of this type of levee is to provide the same level of flood risk management, reduce wind-wave energy, and create a transition zone for wetland habitat benefits.

Sheet pile is planned to be constructed in the levee foundation for all levees, however, it is planned to be limited to two feet above the base of the levee and vinyl sheet pile, which would not withstand breach flows.

Figure 48 shows the feasibility-level typical sections for

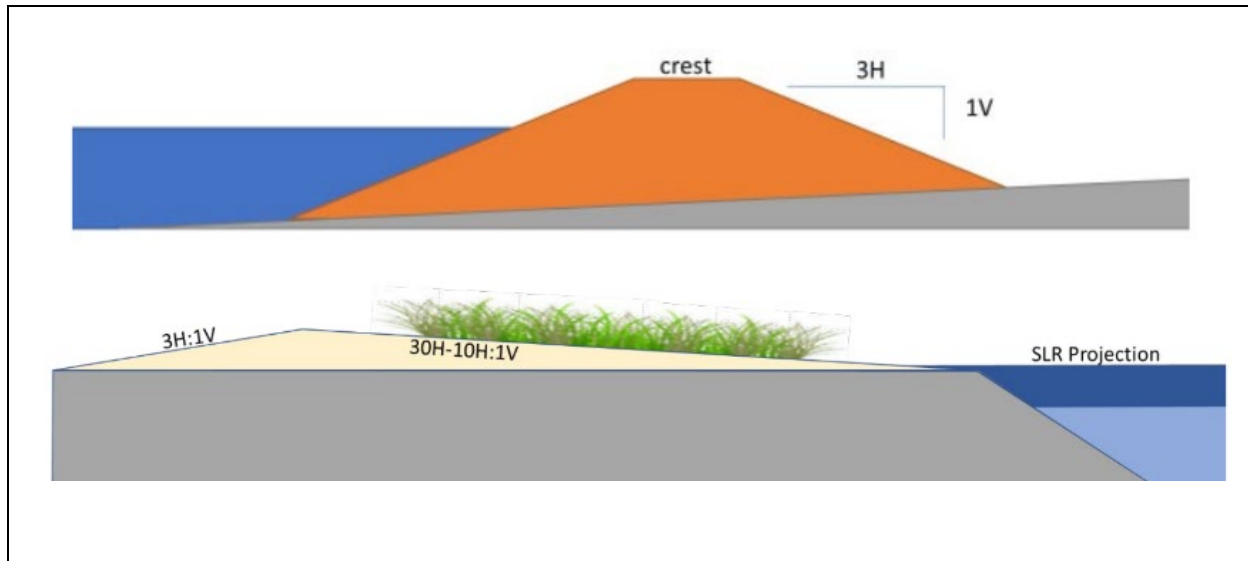


Figure 50. Typical levee section and typical Ecotone levee section

The minimum top of levee elevation in the year 2090 is expected to be at 13.5 ft NAVD88 (Reach 3). The mean still water level (SWL) frequency curve was developed by taking the average of the high RSLC estimate (5.5 ft) and the low RSLC estimate (0.6 ft) for a total of 3.05 ft higher water levels than experienced in 1992. The expected overtopping frequency is approximately 2.2E-03, or 1/450 AEP, as shown in Table 43.

Table 42. Reach 3 Still water level frequency relationship for various RSLC scenarios in 2090

AEP	0.1 (1/10)	0.02 (1/50)	0.01 (1/100)	2.2E-03	1.6E-03	1.09E-03	5.5E-04
1992 SWL (+0 ft)	8.67	9.32	9.63	10.45	10.95	11.45	12.45
2090 low SWL (+0.6 ft)	9.27	9.92	10.23	11.05	11.55	12.05	13.05
2090 intermediate SWL (+1.4 ft)	10.07	10.72	11.03	11.85	12.35	12.85	13.85
2090 high SWL (+5.5 ft)	14.17	14.82	15.13	15.95	16.45	16.95	17.95
2090 mean SWL (high-low avg)	11.72	12.37	12.68	13.5	14	14.5	15.5

While the water levels are likely to be driven primarily by tide and low pressure, some amount of waves generated from wind would likely coincide with the peak water levels. However, the typical low pressure systems may generate opposing winds that directionally generate waves away from the project or the waves may not be severe. The design levee height accounts for coincident 2-foot significant wave height waves with peak water levels. At least by some amount, waves would likely contribute to some additional overtopping erosion, even if only minimally.

The expected duration of overtopping is based on up-scaling of typical tidal durations. For a storm tide resulting in 0.5 feet of overtopping, the total duration of overtopping flows is expected to be less than 3 hours. For 2 feet of overtopping, the duration is expected to exceed 6 hours. Figure 49 shows these expected durations.

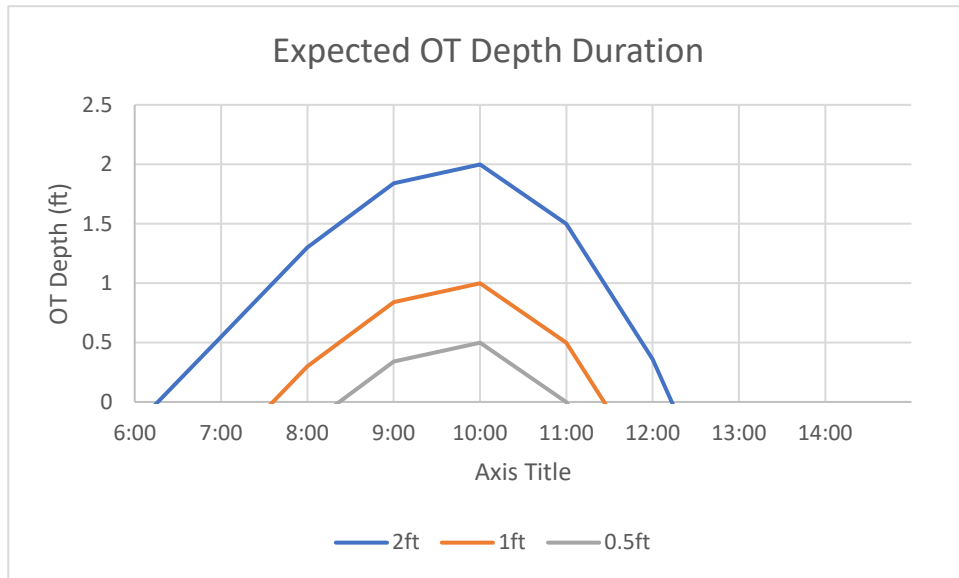


Figure 51. Expected durations of overtopping depths

Hughes and Thornton's research on time-varying overflow of grass-covered slopes recommends tolerable limits for a 1V:3H levee slope with average, good, or "more robust" grass cover. The tolerable limits for depth and duration are shown in Figure 50, compared to the project depth-duration pairs for 0.5, 1, and 2 feet of overtopping. Note that when the depth-duration is above each of the respective lines, the tolerable limit is exceeded.

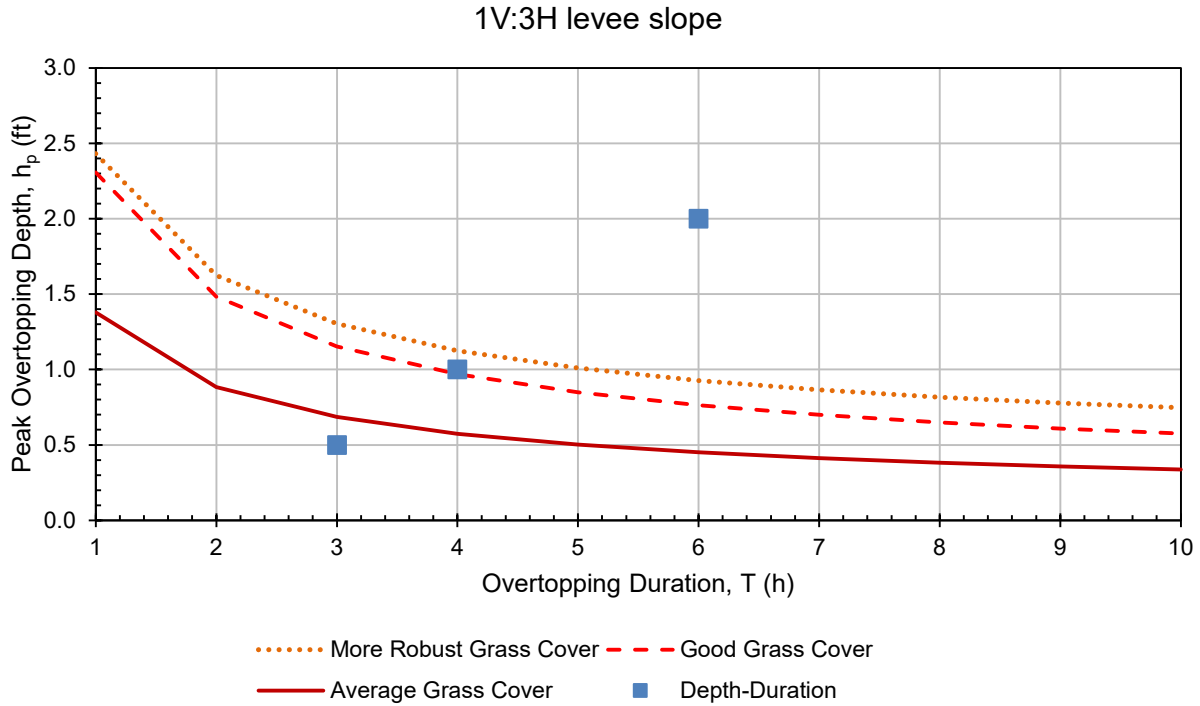


Figure 52. Tolerable limits of time-varying overflow for grass-covered slopes, and expected depth-duration relationship for overtopping events

7.3.3. Performance

There are thousands of levee segments in the Bay-Delta area. Given the varying heights and overtopping frequencies and some of these systems having limited consequences from overtopping, it is possible that some have overtopped and breached and some have overtopped and not breached. There are dozens of levee systems within the San Francisco Bay area and there are no documented levee failures that resulted in life safety risk.

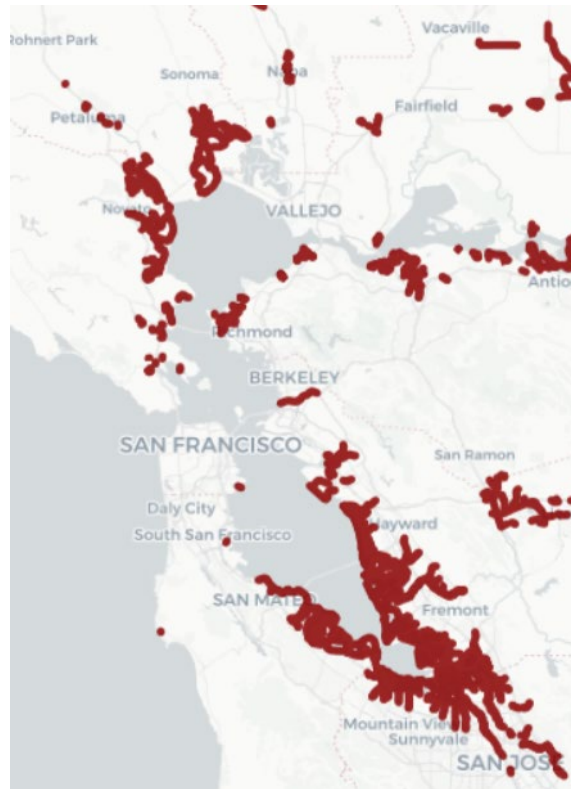


Figure 53. Expected durations of overtopping depths

7.3.4. *Intervention*

Given the low probability of water elevations exceeding the top of the levee in the future, it is unlikely that a temporary levee raise plan will be well developed. Intervention efforts, if any, would be closer to unplanned flood-fighting efforts (e.g., sandbags filled and placed by the community). Currently, the Port has limited experience providing intervention during high water, placing sandbags around buildings and putting out warning signs.

7.3.5. *Likelihood of Failure*

Table 43 summarizes the likelihood factors that make the PFM more likely or less likely to lead to failure.

Table 43.

Likelihood factors for PFM 1.

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
Overtopping durations up to 2 feet are around 6 hours, which may be sufficient duration to fail robust grass cover	Overtopping durations up to 1 foot are less than 4 hours, which may be insufficient duration to fail robust grass cover
Some waves are likely present, which could contribute additional erosion beyond the overflow erosion from still-water.	Levee is expected to be constructed of lean clay, which is less erodible to headcut initiation and continuation.

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
Intervention is possible, but would require around three miles levee raising, which would take a coordinated effort during an extreme regional flood event.	Grass is expected to be well-maintained
Vinyl sheet pile that is primarily in the foundation would do little to resist continuation of erosion.	Some areas of the levee will be hardscaped, which could be less susceptible to erosion
	Some parts of the levee will have flatter landside slopes which would result in lower velocities and a wider embankment to erode.

NOTE: Key factors that drove the likelihood of failure are shown in **bold**.

The annual probability of failure was estimated between 1E-04 and 1E-03 (with and without intervention)

7.3.5.1. Rationale

The team judged the more traditional levee to be the critical section rather than the ecotone levee, due to the narrower levee section having a shorter path and the steeper side slopes having higher velocities.

The team judged the probability unsuccessful intervention to be neutral for less than 0.5 feet of overtopping, as it would likely require minimal sandbagging in areas with minor settlement. For 1-2 feet of overtopping, intervention would likely require miles of sandbagging and may be likely to be unsuccessful.

For low overtopping depths up to 0.5 feet, the team judged a neutral probability of grass cover failing due to the short duration and low velocity. For greater depths, the velocity/shear stress is likely sufficient to cause vegetation damage at the peak of the event.

Headcut initiation of the clay material is unlikely to neutral for low depths and high for overtopping depths of 1-2 feet. Given that the levee is expected to have a relatively uniform elevation, overtopping of miles of levee is likely to expose at least one more vulnerable location to initiate erosion.

The sheet pile would not go up to the crest elevation, so a failure path would still exist above the sheet pile. The team judged mitigating features to arrest the erosion as virtually impossible.

A full breach of the levee was estimated to be unlikely for low overtopping depths and likely for high overtopping depths, as the associated duration of 6 or more hours may be sufficient to fully erode a relatively narrow levee section.

The annual probability of failure (APF) was estimated from a quantitative risk analysis (QRA) using an event tree approach. The overtopping scenarios discussed in Chapter 6 define a continuous loading relationship between peak water surface elevation and AEP. Z-variate interpolation was used to calculate the AEP for peak water surface elevations between those that define the relationship. The conditional probability of failure was estimated as a function of overtopping depth by team elicitation.

The estimated APF for overtopping with breach is estimated to be 3.0E-04 failures per year with and without intervention. Therefore, the estimated APF is between 1E-04 and 1E-03 failures per year with and without intervention. Calculations are provided in Appendix B of this report.

Table 44.
System response curve for PFM 01.

Overtopping Depth (feet)	System Response Probability (with intervention)	System Response Probability (without intervention)
0	0	0
0.5	0.004	0.009
1.0	0.202	0.225
2.0	0.601	0.607

7.3.5.2. Uncertainty

There are numerous uncertainties in the overtopping breach risk estimate. The relative sea level change could be multiple feet higher or lower than the expected value in the year 2090. Waves are not explicitly estimated in association with this type of high water event, so they could be negligible or significant depending on the wind conditions. The exact design of the levee is still early in the process, so a different levee section than the proposed one evaluated may have a different risk estimate. The condition of vegetation, future modifications (e.g., 408 permits for infrastructure in or near the levee section), and potential rainfall erosion is unknown.

7.3.6. **Incremental Life Loss**

The average incremental life loss was estimated between 0.01 and 0.10.

7.3.6.1. Rationale

Consequence modeling for the 2090 condition indicates zero life loss for both breach and non-breach overtopping scenarios. Given the low depths of inundation, the likely chance that there would be awareness and warning of the high water event, and the ability to reach high ground over a short distance (by vehicles or by foot), life loss is expected to be non-zero but less than one.

7.3.6.2. Uncertainty

Future population characteristics are unknown as are the population response to predicted high storm tides. While impacts to the population and their protective action initiation are uncertain, life loss is expected to be low for most scenarios due to the relatively low flood depths.

7.3.7. **Incremental Economic Cost**

The average incremental economic cost was estimated between \$10M and \$100M.

7.3.7.1. Rationale

The expected annual damages without the project in place by the year 2090 is estimated to be hundreds of millions of dollars per year. With the project in place and a breach due to overtopping, the breach inundation is expected to be limited as compared to the without project condition due to the short duration of peak storm tides and the limited length of breach flow compared to widespread tidal exceedance of existing features. A breach occurring in different reaches of the system is expected to have slightly different economic damages, but all scenarios result in damages between \$10M and \$100M for loading near the top of the levee. For breach and non-breach scenarios that exceed the top of levee, damages may

approach billions of dollars due to wide spread overtopping. These events are expected to be infrequent and incremental damages directly attributed to breaching are expected to be much less than the total damages.

7.3.7.2. Uncertainty

Future investment in this area is uncertain. With the project in place, the structure inventory in place may incorrectly assume that flood risk has been eliminated and have higher values of structures and contents. The project is expected to perform well in reducing average annual damages to the area, but risk still remains, especially for overtopping risks if relative sea level change is greater than expected over the life of the project.

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Appendix A: Excluded Potential Failure Modes

The following sections summarize the potential failure modes that were excluded from further consideration because they were deemed non-credible or credible but non-risk drivers.

A.1. Seismic PFM Exclusions

The following paragraphs are used to justify exclusion of PFM's 7, 31, 42, 52, and 65.

Seismic events can occur at any time and are inherently unpredictable. Generally, the coincident probability of a seismic event occurring simultaneously with a flood is low, and the San Francisco Waterfront project is no exception. The current TSP indicates that the project will be designed to survive a 950-year seismic event through seismic ground improvement. However, the current level of design does not provide sufficient detail to fully evaluate any seismic-related failure modes. Some general conclusions can be made that apply to all seismic failure modes, which serve as the rationale for their exclusion from this risk assessment. Once the design has progressed and details on seismic ground improvement are developed, all seismic failure modes should be reevaluated.

With any risk assessment, the results are a snapshot in time. As our understanding of seismicity and structural response change over time the estimated risk of the project will change. Subsequent paragraphs are based on a limited design with our current understanding of seismicity and structural response.

A.1.1. *Loading.*

To properly calculate the loading the duration of the tidal events must be considered along with the probability of seismic loading. A normal tide cycle that occurs twice every day with a typical duration of 6 hours was assumed; it must be acknowledged that this duration is likely overstated by a factor of 2 or more due to the effect of transitioning from low tide to high tide and vice versa. Discussions with the PDT indicated that the three SLC curves have an equal probability of occurring. As such, the probability of loading does not change between curves. For the 2090-year plan the 1% AEP event was selected. For the 2140-year plan the 99% AEP event was selected.

A.1.2. *Assumed structural response probability.*

The current TSP indicates that the project is being designed to survive a 950-year seismic event. While damage is expected at this level of seismic activity, catastrophic failure of the system is not anticipated. Localized repairs (intervention) will likely be necessary before any subsequent flood event. For seismic loadings exceeding the 950-year event, the project is likely to suffer significant damage and potentially fail. The current TSP indicates that all features will be designed to remain elastic under the 144-year seismic event and survive the 950-year seismic event.

For the realization of consequences (breach) to occur a subsequent flood event must occur. For the year 2090 and the 1% extreme water level, there is approximately two (High SLC Curve) to five (Low SLC Curve) feet of freeboard, for the year 2140 it is assumed any second action will maintain the difference between the high SLC curve and top of protection, therefore for the 0.99 probability event the freeboard is

approximately 3.1 feet. The line of protection must be reduced by greater than the freeboard amount to result in inundation of the interior. Since a subsequent high-water event is needed to flood the interior intervention is possible for all scenarios.

Assuming, current design standards are followed, and construction receives adequate QA/QC, the SRP without intervention for a 950-year is conservatively assumed to be 0.1. Once the design seismic loadings have been exceeded sufficiently the SRP will trend towards virtually certain (0.999) to fail. The exact seismic loading responsible a virtually certain failure is unknown and beyond the scope of this risk assessment to determine. The 10,000-year event has been chosen as an analog for that virtually certain failure SRP without intervention, and linear interpolation will be used to determine different SRP values. The resulting SRP for a 950-year seismic event is 0.1 and for a 2450- year seismic event is 0.25. Since the design assumes that during the 144-year event the project features will remain elastic the SRP for the OBE is assumed to be 0.001 or virtually impossible to fail, see Figure 51 for the SRP curve.

Please note SRP's were developed without expert elicitations, adequate design drawings, or calculations. As such, all seismic failure modes must be reevaluated once design has progressed.

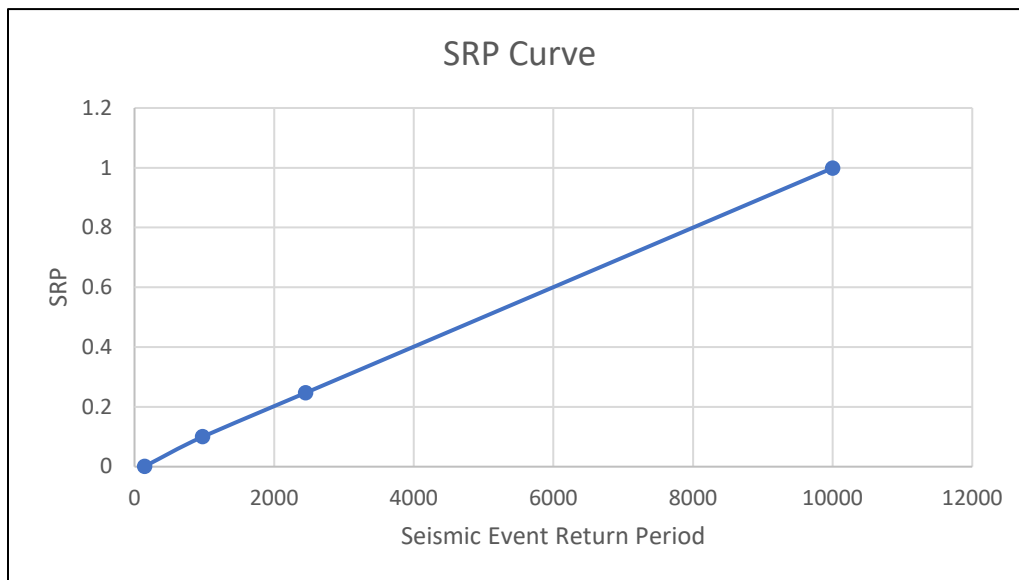


Figure 54. Assumed SRP Curve with respect to seismic return period.

A.1.3. APF.

Using an assumed emergency repair (intervention) window, the APF for the seismic event, and the adjusted flood hazard, we can calculate probability of a flood prior following a breach using the RMC Event Combination toolbox. The time between a breach due to a seismic event and a subsequent flood, allows for temporary repairs. Temporary repairs (intervention) can include several different methods of repair; the most common are sandbags, soil baskets, and jersey barriers.

For the year 2090: Assuming a 1% AEP flood hazard and adjusting for duration exceedance, a 975-year seismic event, and a 365-day temporary repair window, the probability of flooding following a breach is between 1E-07 and 1E-06, or 3 orders of magnitude (OOM) below overtopping with breach.

For the year 2140: Assuming a 99% AEP flood hazard and adjusting for duration exceedance, a 975-year seismic event, and a 7-day temporary repair window (a 7-day repair window was assumed, however the

San Francisco Waterfront Storm Damage Risk Reduction System

actual window is likely between 1 and 7 days based on the frequency of flooding), the probability of flooding prior to completion of temporary repairs is between 3E-07 and 3E-06, or 2 orders of magnitude (OOM) below overtopping with breach.

APF	Time Between Flood Exceeding Deformed LoP and Seismic Event							Year Seismic Event	SRP
	1	7	14	30	90	180	365		
6.90E-06	9.45E-11	6.61E-10	1.32E-09	2.83E-09	8.50E-09	1.70E-08	3.44E-08	145	0.001
8.89E-05	1.22E-09	8.52E-09	1.70E-08	3.65E-08	1.10E-07	2.19E-07	4.43E-07	225	0.02
9.26E-05	1.27E-09	8.88E-09	1.78E-08	3.81E-08	1.14E-07	2.28E-07	4.62E-07	475	0.044
1.03E-04	1.40E-09	9.83E-09	1.97E-08	4.21E-08	1.26E-07	2.53E-07	5.12E-07	975	0.1
1.01E-04	1.38E-09	9.69E-09	1.94E-08	4.15E-08	1.24E-07	2.49E-07	5.04E-07	2,475	0.25
9.99E-05	1.37E-09	9.58E-09	1.92E-08	4.10E-08	1.23E-07	2.46E-07	4.98E-07	10,000	0.999

Figure 55. Probability of flooding following a breach 2090 plan.

APF	Time Between Flood Exceeding Deformed LoP and Seismic Event							Year Seismic Event	SRP
	1	7	14	30	90	180	365		
6.90E-06	9.35E-09	6.52E-08	1.30E-07	2.75E-07	7.93E-07	1.49E-06	2.69E-06	145	0.001
8.89E-05	1.21E-07	8.40E-07	1.67E-06	3.55E-06	1.02E-05	1.93E-05	3.47E-05	225	0.02
9.26E-05	1.26E-07	8.76E-07	1.74E-06	3.70E-06	1.07E-05	2.01E-05	3.62E-05	475	0.044
1.03E-04	1.39E-07	9.70E-07	1.93E-06	4.09E-06	1.18E-05	2.22E-05	4.01E-05	975	0.1
1.01E-04	1.37E-07	9.55E-07	1.90E-06	4.03E-06	1.16E-05	2.19E-05	3.95E-05	2,475	0.25
9.99E-05	1.35E-07	9.45E-07	1.88E-06	3.99E-06	1.15E-05	2.17E-05	3.90E-05	10,000	0.999

Figure 56. Probability of flooding following a breach 2140 plan

A.1.4. Consequences.

Consequences are assumed as the same order of magnitude for both life loss and economic damages as overtopping with breach when accounting for pre-flood seismic consequences (non-breach). As part of future risk assessments, an estimation of pre-flood seismic consequences must be made to properly estimate the flood risk due to seismic driven failure modes.

A.1.5. Rationale for exclusion.

For the year 2090: Exclusion of seismic failure modes are primarily based on two factors. The first and most significant is flood related incremental consequences are low and are assumed as the same OOM as overtopping with breach. The probability of flooding following a breach is between 1E-07 and 1E-06 or 3 OOM below overtopping with breach.

For the year 2140: The probability of flooding following a breach is between 1E-07 and 1E-06 or 2 OOM below overtopping with breach. Since consequences are assumed as the same OOM as overtopping with breach, the average annual life loss is also 2 OOM lower.

Based on the information presented above, all seismic failure modes are considered non-risk drivers for this risk assessment but should be reevaluated once the design has progressed. Additionally, an estimation of pre-flood seismic damages must be made to properly calculate consequences for seismic related failure modes.

A.1.6. Recommendations.

The following recommendations are based on the many assumptions made in the development of the failure mode and do not account for any ancillary benefits from the seismic mitigation measures that may be part of the combined net benefits plan.

The current TSP includes a significant seismic ground improvement effort. While these improvements reduce the seismic risk to the project, they do little to address the total incremental flood risk, which is dominated by overtopping with breach. The current seismic mitigation solutions reduce the seismic-related failure probability by approximately 3 orders of magnitude compared to the overtopping with breach probability for the 2090 plan, and by approximately 2 orders of magnitude compared to the overtopping with breach probability for the 2140 plan. It would be more appropriate to target a 1 to 1.5 order of magnitude reduction in APF below overtopping with breach. Even if the targeted reduction cannot be achieved, any percentage reduction in construction costs is likely on the order of hundreds of millions, if not billions of dollars.

We recommend revising the extents and types of seismic ground improvement and/or project features to provide a cost-effective seismic mitigation solution that is solely designed for the benefit of the levee system that reduces flood risk. Ancillary benefits may be realized but must not be the intent of the solution. Additionally, reevaluate all seismic failure modes once the design has progressed and consider including an estimation of pre-flood seismic consequences (non-breach) in the reevaluation, as well as post-earthquake response and repair costs IAW ER 1110-2-1806 (2024) Section 9.7.c.

Alternative	2040 Target Performance	2040 Finish Elevation	2090 Target Performance	2090 Finish Elevation
Alternative A	No Action			
Alternative B	Floodproof areas at risk of 1% AEP coastal flooding; retreat areas at risk of monthly coastal flooding; add assets as risk increases over time.			
Alternative C	1.5' SLC	13.5' NAVD88	N/A	N/A
Alternative D	1.5' SLC	13.5' NAVD88	3.5' SLC	15.5' NAVD88
Alternative E	3.5' SLC	15.5' NAVD88	7.0' SLC	19.0' NAVD88
Alternative F	3.5' SLC	15.5' NAVD88	7.0' SLC	19.0' NAVD88
Alternative G	3.5' SLC	15.5' NAVD88	7.0' SLC	19.0' NAVD88

Figure 57. First and second action elevations from the January 2024 Draft Feasibility Report.

Table 2: Conceptual Framework for TSP Subsequent Actions				
Reach	First Action	Second Action Low RSLC	Second Action Intermediate RSLC	Second Action High RSLC
Fisherman's Wharf, Reach 1	Alternative B	No action	Alternative B (Additional NS)	Alternative G 19'
Embarcadero, Reach 2	Alternative G 15.5'	No action	No action	Alternative G 19'
South Beach/Mission Bay, Reach 3	Alternative D 13.5'	No action	Alternative D 15.5'	Alternative E 19'
Islais Creek/Bayview, Reach 4	Alternative D 13.5'	No action	Alternative D 15.5'	Alternative E 19'

Figure 58. Conceptual framework for TSP Subsequent Actions from the January 2024 Draft Feasibility Report.

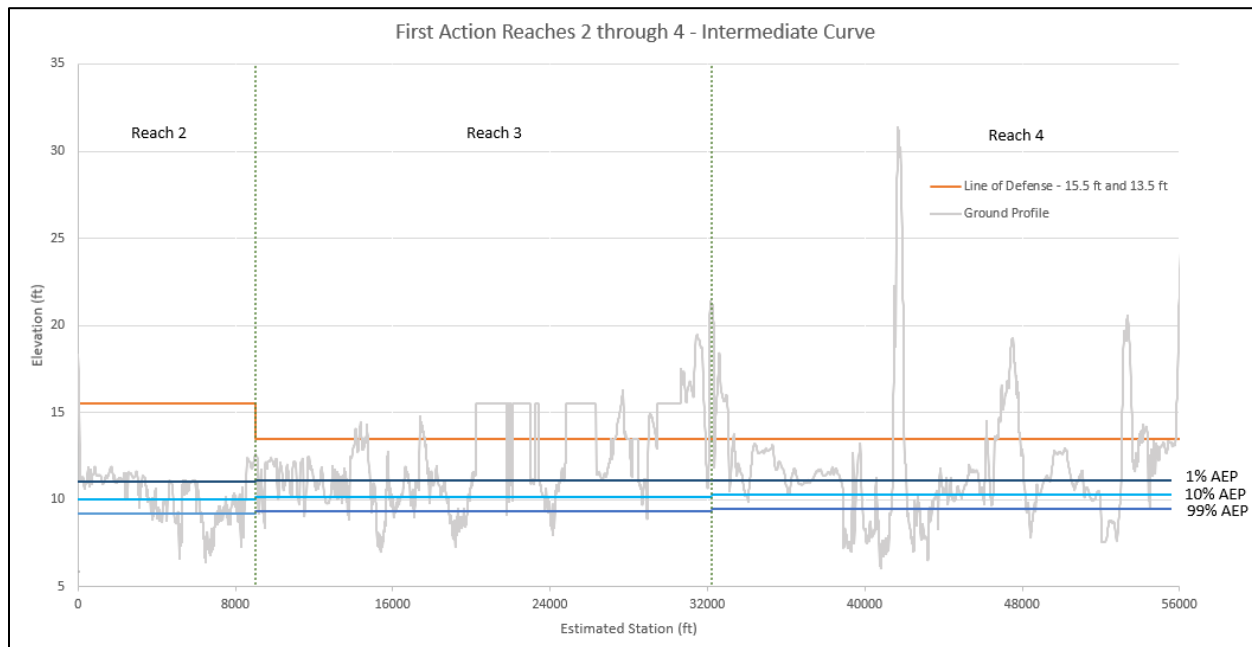


Figure 59. First action profile for reaches 2 through 4 using the intermediate sea level change curve.

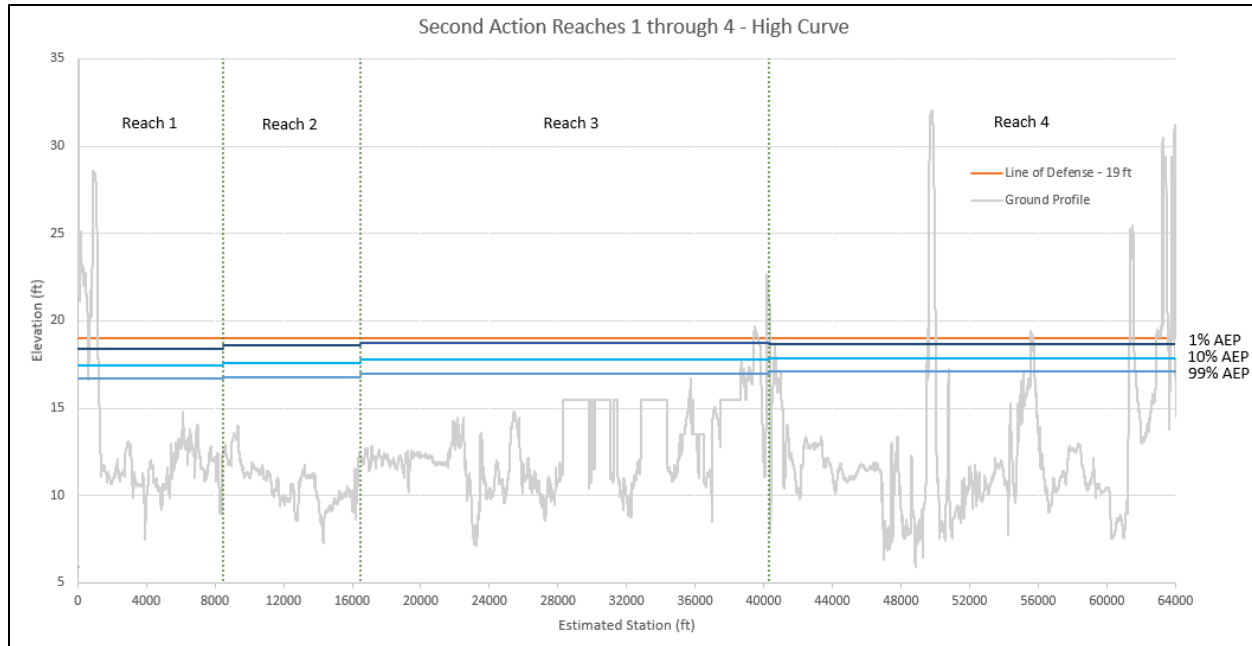


Figure 60. Second action profile for reaches 1 through 4 using the high sea level change curve

A.2. PFM 02: Backwards erosion piping (BEP) through the foundation

This PFM is regarding backward erosion piping (BEP) through the foundation material underneath the levee. As the sea water surface elevation rises, seepage occurs in a continuous (waterside to landside) zone of uniform sand beneath the levee embankment. An unfiltered exit must exist at the landside toe or at some distance away from it on the landside. Sufficient seepage forces detach soil particles in the uniform sand layer at the seepage exit, initiating backward erosion piping. The overlying materials support a continuous and stable roof above the developing pipe. Sufficient horizontal gradients exist for progression of a pipe towards the sea. If detection and intervention are unsuccessful, BEP continues until the pipe reaches the waterside face. Gross enlargement of the pipe occurs leading to instability and collapse of the embankment, lowering the available freeboard below the water surface elevation. Erosion continues to full breach development and uncontrolled release with associated consequences.

The base of the levee can act as a roof. Levees will be constructed in Reaches 3 (South Beach / Mission Bay) and 4 (Islais Creek / Bayview) to an elevation of 13.5'. The Risk Assessment team does not have geotechnical data from the area where the levees will be constructed. However, there is data from Reaches 1 (Fisherman's Wharf), 2 (Embarcadero), and a portion of Reach 3. Figure 58 shows the subsurface along the existing seawall from Fisherman's Wharf to Mission Bay. The stratigraphy below the fill is defined with their correlation to glacial-interglacial stages: Young Bay Mud (YBM), Upper Layered Sediments (ULS), Old Bay Mud (OBM), Lower Layered Sediments (LLS), and Franciscan Complex bedrock (KJf). The fill is heterogeneous sandy fill that consists of dune sand and debris. There are two additional subunits for YBM: YBM-clay and YBM-sand. The former is made of soft to firm clays with some sands and the latter has loose to firm sands (including poorly-graded sand) and silts occurring as interbeds. On the other hand, ULS varies in general from medium dense to very dense poorly-graded

sand to medium stiff to very stiff clay (fat and lean). Figure 59 show conceptual Geologic Model for South Beach. This generalized model is the closest to Reaches 3 and 4.

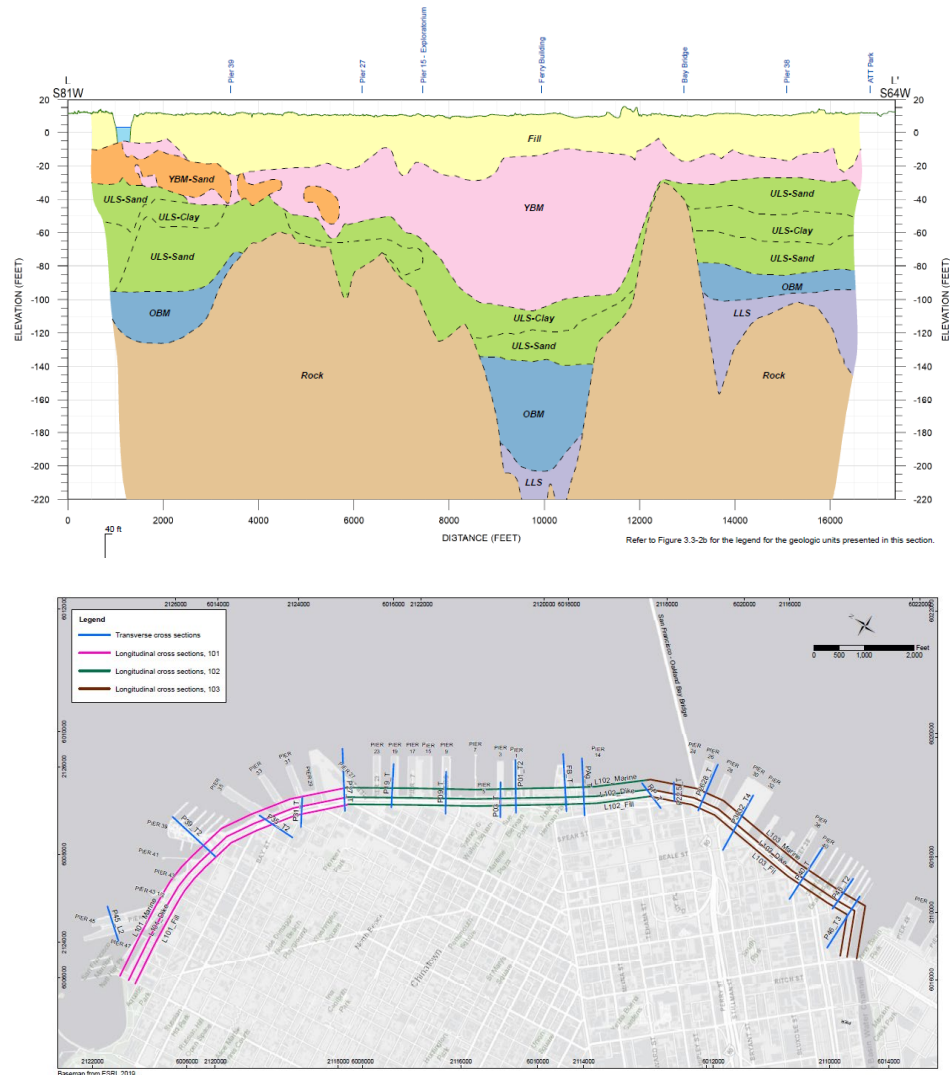
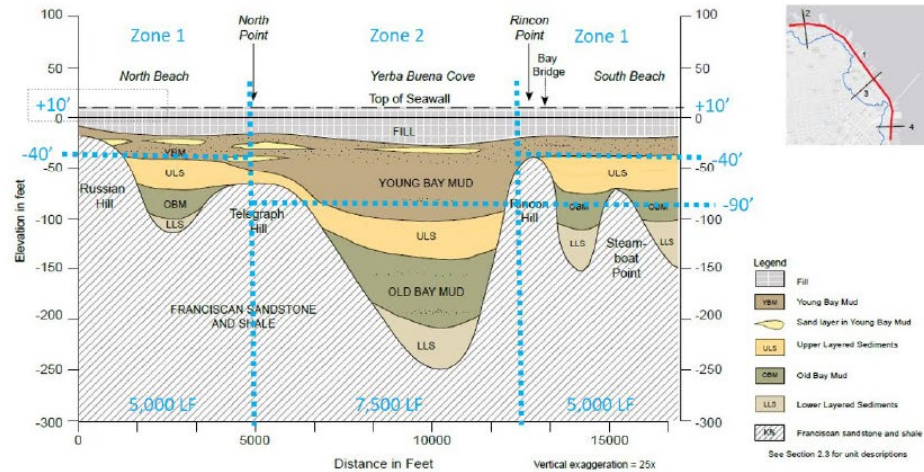


Figure 61. Subsurface along existing seawall.

Geotechnical Zones- NWF

Zone 1 – 50' wide x 50' deep improved soil buttress
 Zone 2 – 100' wide x 100' deep improved soil buttress



Source: CH2M/Arcadis Team 2020c

Figure 62. Conceptual geologic model near South Beach.

As shown in the figures, there is a lot of variability in the soils where the levees will be built. There could be layers of poorly-graded or pipeable sands that could be discovered when drilling for the geotechnical study. Borings will help to determine if a continuous path from waterside to landside could exist where the levees will be built. In addition, the levee could provide a roof for a seepage path within the foundation. Conditions for BEP could be possible if an unfiltered exit exists landward from the levee. This is why it is very important to conduct seepage modeling and ensure that the seepage criteria for BEP is met in accordance with Engineering Manual (EM) 1110-2-1913 Design and Construction of Levees (USACE 2000 or newer version). The current plans are for any levees taller than 4 feet to have sheet piles that penetrate through the foundation under the levee. Considering that levees will have sheet piles that would cut off a continuous path and will meet the criteria for BEP, the team has determined that this PFM is not a risk driver.

A.3. PFM 03: Concentrated leak erosion through a crack in the embankment

This PFM is concerning a possible continuous crack in an embankment levee in which concentrated flow could erode and lead to breach. This type of internal erosion is called concentrated leak erosion (CLE). As the sea water level rises, seepage through a crack can detach and transport material. Continuous erosion can lead to breach if there is no intervention. Levees will be constructed in Reaches 3 and 4 to an elevation of 13.5'. The levees will tie into floodwalls, existing levees, high ground or other structures to achieve the elevation. Levees would be constructed of imported impervious earthen fill material, assumed to be procured from a commercial source near San Francisco. The levees would be constructed by compacting lifts using heavy machinery (i.e., bull dozers, track hoes, roller compactors, etc.). The feasibility study indicates that slopes will not be steeper than 3H:1V (Figure 60), and isolated areas could have slopes ranging from 5H:1V to 12H:1V. Levees will be designed in accordance with Engineering

Manual (EM) 1110-2-1913 Design and Construction of Levees (USACE 2000 or newer version). As the levee is raised, there could be cracking during its construction or when settling. The embankment is expected to be of similar material with limited differential settlement being expected. In addition, it is expected that the ground improvements would allow for an uniform levee settlement. Special attention to how the levee is constructed near the tie-ins is very important to avoid preferential paths for water to flow through. There have been no issues related to cracking/settlement in levees in the area. This PFM was determined to be a non-risk driver assuming proper design, uniform material, and good construction methods.

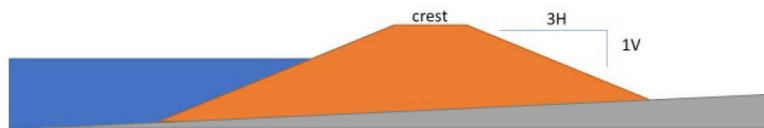


Figure 63. Typical levee section.

A.4. PFM 04: CLE along a pipe penetration in the levee or foundation

This PFM requires a crack or poorly compacted zone along the entire length along a pipe or culvert that would allow for a concentrated leak erosion. Similar to PFM 3, CLE through a crack along a pipe (drainage, water supply, sewer, gas, etc.) can lead to breach. The pipes could either be in the foundation under the levee or through the embankment itself. For this PFM to occur, there needs to be sufficient flow and the hydraulic shear stress should exceed the material critical stress to start movement of particles. As soil particles detach, the crack becomes larger and can lead to failure of the structure.

Existing shallow pipes could be integrated into the levee design if they can be accounted for. However, it is not known how many pipes there could be deep enough in the foundation that may not get consideration. The overall length of the pipes is also unknown. For new pipes, it is expected that fill will be compacted properly around them and filters along their length and landside exit where they could daylight. No distress or failure has been observed in the existing levees during previous loading events that is linked to CLE along the penetrations. Assuming filter zones will be used and long failure paths, this PFM was determined to be a non-risk driver.

A.5. PFM 05: CLE along a tie-in of a levee (with floodwall, with high ground, etc.)

This PFM is concerning a possible continuous crack, void or flaw at tie-ins of a levee with a floodwall, high ground or any other structural feature in which concentrated flow could erode and lead to breach. As the sea water level rises, seepage through a defect can detach and transport material. Continuous erosion can lead to breach if there is no intervention. As mentioned in PFM 3, the levees will tie into floodwalls, existing levees or into high ground to achieve the design elevation in Reaches 3 and 4. Levees will be constructed using impervious earthen fill material and compacted with heavy machinery in lifts. Differing

foundation conditions, differential settlement of the levee, and not adequate construction/compaction methods with new or existing infrastructure can lead to cracking or flaws in which water flow could potentially erode material. Areas that require extra attention are tie-ins of the levees with soils that may contain lightweight materials and geofoam structures that could be more susceptible to CLE. In addition, the new levee needs to be properly embedded and compacted to the other features. This PFM was determined to be non-risk driver due to the assumed proper compaction practices at the tie-ins.

A.6. PFM 06: Hydraulic fracture in foundation leads to CLE

This PFM is regarding hydraulic fracture in the foundation that leads to CLE. For this case, hydraulic fracture occurs when the seepage enter through potential cracks or paths, and the hydraulic pressure is greater than the stresses at rest. This causes crack propagation that can lead to breaching of the levee. This PFM could happen within the foundation of the levees in low stress zones (or tension zones) such as areas with severe differential settlement or foundation anomalies. The foundation could have high variability, as mentioned in PFM 2. A low stress zone could get exposed to high pore pressures as sea level rises. However, this increase would not be too drastic and the ground improvement would likely mitigate the risk. The team has determined that this PFM is not considered a risk driver.

A.7. PFM 07: Seismic event leads to levee crest deformation due to lateral spreading and/or liquefaction

Exclusion discussed in A.1.

A.8. PFM 08: Settlement of the levee leads to more frequent overtopping

This PFM alludes to settlement in some areas along the levees that can result in lowering the levee crest below the design elevation and more frequent overtopping. Vertical deformation of a levee that is improperly designed when settlement is not accounted for can contribute to more recurrent overtopping that could lead to breach of the structure. Chapter 7 discusses overtopping scenarios and associated consequences due to breach. The two main contributors to vertical subsidence of a levee are due to seismic events and geotechnical consolidation settlement. The soils on which levees will be built are mostly fill, Young Bay Mud (soft to firm clay) and Upper Layered Sediments (layered sequence of sands clays and gravels). Some of the layers could potentially settle with the additional weight of the levee. Consolidation settlement can occur across the waterfront and it needs to be considered in the design due to the differing subsurface conditions along all reaches. The ground improvement will stiffen the matrix of the foundation soils and reduce settlement. In addition, overbuilding the levee to a higher elevation where settlement is expected could be necessary. A plan to monitor the settlement is necessary whether ground improvement or overbuild approaches are used to detect if settlement is more than expected. The team has determined that this PFM is not a risk-driver if settlement is incorporated in the design.

A.9. PFM 09: Slope instability leads to loss of levee crest

This PFM is concerning the potential of a slope stability failure along the landside or waterside of the levee. Slope instability leads to the levee crest lowering and water could get into the protected area. If there is no intervention, the levee could breach. Landside or waterside slope stability failures that are not able to be repaired could happen when the levee is not loaded on the waterside slope (low water or sunny day). Conversely, these failures could also happen during an event such as a king tide. The additional pressure on the waterside slope could be a contributing factor. Lastly, water elevation could reduce faster than the levee can drain which can lead to a rapid drawdown failure due to.

There are no slope stability analyses performed for the proposed levees yet. Results of modelling of landside and waterside slope failures need to meet the factor of safety criteria presented in EM 1110-2-1902 (Chapter 3) and EM 1110-2-1913 (Chapter 6, 2000 edition). Scenarios of end-of-construction and rapid drawdown need to be incorporated. The documentation suggests that an “impervious earthen fill material” will be used. Fill of fine-grained materials is less susceptible to rapid drawdown than levees made with sandy materials. The soil shear strength parameters used for the modeling will be selected based on the laboratory testing program from the geotechnical exploration. It is also important to consider the higher shear strength of the soils after the soil improvement. Assuming that the soil strength values for the slope stability analyses will be appropriate for the new fill and underlying materials of the substrata, the team determined that this PFM is not a risk driver. Lastly, it is assumed that any slope issues will be detected early due to the location of this levee, as it is an urban area where people walk and work.

A.10. PFM 10: Wave erosion progresses to loss of levee crest

For at least some locations along the alignment, there will be conditions over the life of the project where storm tides load the levee embankments and large waves will be present. If large waves persisted for long durations, enough wave erosion could occur that the levee becomes unstable and the crest of the levee is lost. Given that the levees will be constructed of clay material and have well-vegetated cover, they are likely able to withstand storm tide durations of wave erosion without significant damage to the point of a full breach. Also, the design of the levees is such that the embankment is sufficiently wide and may be higher in elevation than water levels that are associated with the highest wave conditions, so erosion may be unlikely. Additionally, any areas that do have wind fetches with higher wave exposure and more potential for wave erosion will be assessed during design and armored appropriately with riprap or other mitigating measures. All features should be inspected through routine inspections to identify areas of erosion throughout the project life.

A.11. PFM 11: Wave overtopping erosion of the embankment

Wave overtopping should be considered in the design of the recommended design grade elevation to ensure limit states for different features are not exceeded. The design should have high assurance that risk of wave overtopping erosion breach should be lower than overtopping/overflow from high storm tides.

A.12. PFM 12: Global instability of the levee embankment and foundation

This PFM is concerning the potential of a global stability failure of the levee and its foundation leading to the levee crest lowering and water getting into the protected area. If there is no intervention, the levee could breach. A failure along a deep slip surface could mobilize the entire levee. This PFM is similar to PFM 9 for slope stability failures. There are no slope stability analyses performed for the proposed levees yet. Results of modelling of a global stability failure toward landside or waterside slope need to meet the factor of safety criteria presented in Chapter 3 of EM 1110-2-1902 (USACE, 2003) and Chapter 6 EM 1110-2-1913 (USACE, 2000). Scenarios of end-of-construction and rapid drawdown need to be incorporated. The soil shear strength parameters used for the modeling will be selected based on the laboratory testing program from the geotechnical exploration. Assuming that the soil strength values for the slope stability analyses will be appropriate for the new fill and substrata, the team determined that this PFM is not a risk driver.

A.13. PFM 13: Animal burrows leads to shortened seepage path for BEP or CLE

This PFM is regarding a failure of the levee due to concentrated leak erosion (CLE) or backward erosion piping (BEP) that initiates in a single or a series of animal burrows. It is assumed for this PFM that there is continuous erosion starting in an animal burrow and through the embankment or its foundation as sea water level rises leading to breach. The most common burrowing rodents that could be found in this location are gophers, but they are not common. There are no previous incidents of animal burrows in the existing levees. Currently, there is no operation and maintenance plan in place to control animal activity. In addition, the levee will be inspected properly (frequency to be determined) and animal activity can be detected during these inspections. Furthermore, levee maintenance personnel or residents could also notice if extensive burrowing is occurring on the slopes. It is assumed that levees will be well maintained. Mitigation measures can be implemented if necessary, such as sandbags, if erosion is noticed due to animal activity. Even if the unlikely scenario of animals digging into the levee happens, this event requires many nodes in an event tree that would lead from initiation to breach of the levee. Due to all these reasons, the team concluded that this PFM is not a risk driver.

A.14. PFM 14: Floatation of lightweight material in tie-in (high ground) leads to seepage and stability mechanisms that results in flanking of the levee

This PFM is concerning a possible continuous crack, void or flaw at tie-ins of a levee with high ground in which concentrated flow could erode and lead to breach. As the sea water level rises, seepage through a defect can detach and transport material. Continuous erosion can lead to breach if there is no intervention. As mentioned in PFM 5, there are high ground areas that have been constructed already that use lightweight material. The incompatibility of these materials with respect to the levee could cause issues and lead to preferential seepage paths. Levee fill should be properly embedded and compacted to provide a seamless transition. This PFM was determined to be non-risk driver due to the assumed proper compaction practices at the tie-ins.

A.15. PFM 15: CLE along vegetation roots beneath the levee

In addition to cracks or conduits, CLE could potentially happen through roots of vegetation, especially trees, shrubs or other plants with long roots. The roots could a weak spot and provide preferential paths for seepage through the levee or its foundation. During a high water event, the flow of water could initiate erosion that could just like through cracks or other defects. Continuous CLE could lead to breach of the levee. The risk assessment team is aware that the project has an Engineering with Nature component. This means that the project will likely incorporate natural and nature-based features (NNBF) that have some benefits such as provide some habitat for endangered species and connectivity to nature. For example, part of the design will consider ecotone levees. This type of levees provides a transition zone toward tidal marsh with native plants on the waterside and with gentle slopes. They also could provide coastal storm risk reduction benefits and wave dissipation. These slopes can vary from 10H:1V to 30H:1V as seen in Figure 61.

The USACE policy has been to keep levees “vegetation free” on both slopes, crest and 15 feet away from the landside and waterside toes. The recommended vegetation cover for the levees has always been mowed grass. This is due to the uncertainty of how trees, shrubs and other vegetation could interact with the levees. Also, vegetation makes it harder for inspections and intervention. However, the USACE has recently adopted risk-informed design process that could allow vegetation other than grass on levee, mainly on the waterside slope and waterside toe. This process requires evaluation of what type of vegetation will be planted and its associated risks. A waiver would be needed allow for vegetation, which depend on the results of the risk assessment. In addition, the vegetation maintenance needs to be properly accounted in the O&M activities. This PFM was determined to be a non-risk driver, but if the project incorporates vegetation into the levee design, a risk assessment with an engineering analysis is required in the future to understand the risks and to determine if it will not have a negative on its performance.

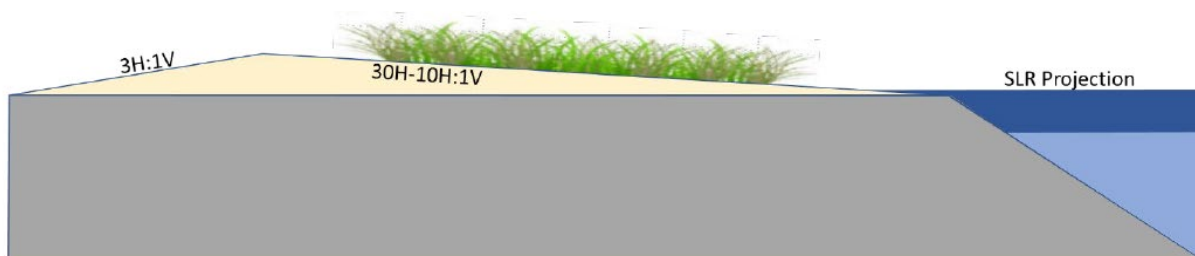


Figure 64. Typical cross section of ecotone levee.

A.16. PFM 16: Waterside erosion of the levee embankment leads to loss of crest

As water moves around the San Francisco Bay and through Islais and Mission Creeks, higher velocities would present the potential for erosion of earthen features and the associated foundations. The bay can experience higher tidal currents, but primarily through the narrower Golden Gate area and not along the project alignment. The creeks are associated with relatively small drainage areas, so it is likely that velocities from tidal fluctuations are greater than runoff flood flows. For all conditions, these velocities

are likely lower than critical erosion values for embankment and foundation material. If any areas are expected to experience higher velocities, they should be armored during design.

A.17. PFM 17: Wake or prop wash scour leads to levee breach

The San Francisco Bay and the Port of San Francisco have a long history of shipping traffic, including deep-draft navigation of large vessels capable of producing strong wake. Other moderately-sized ships and tug-boats can also produce “prop wash”, or the turbulence directed off of the propeller, when maneuvering close to the port, which has the potential to cause erosion. The design should account for all potential sources and magnitudes of wake produced by the typical traffic within the bay. Appropriate erosion mitigation should be included in design, where appropriate.

A.18. PFM 18: Internal erosion out of a pressurized pipe

For this PFM, pressurized pipe ruptures within or under the levee. Escaping fluid erodes the embankment sufficiently that the crest is impacted and eventually lowered below the flood water level. Overtopping occurs through the eroded area and leads to breach of the embankment.

The Risk Assessment team was informed that there are drinking water and emergency fire water system pipes in the ground where the levees will be constructed. Size and lengths of these pipes can vary. The San Francisco Public Utilities Commission is the primary public agency that distributes water. The emergency fire water system is called the Auxiliary Water Supply System and is independent from the municipal water supply. To cause major issues to the levee that could lead to its breach, a rupture of one of these pipes would have to go unnoticed. Loss of pressure due to rupture could be independent from low water or high water events. As these pipes operate constantly and are heavily monitored, the likelihood of any of them experiencing a major rupture that could cause erosion issues to the levee are considered remote. The loss of pressure would be identified by the public agencies and would have to address the issue quickly. The team recommends valves on both sides of the levees for pressurized pipes that go through the levee embankment or its foundation. The team has determined this PFM is not a risk driver.

A.19. PFM 19: Internal migration into parallel storm sewer

Internal migration into a culvert (also known as stoping) occurs when soil material erodes into an open crack or defect of a culvert. The soil particles migrate into the pipe and a void at the interface between the pipe and the soil starts growing. As more material falls into the pipe along with the flow of water, a sinkhole can develop, and overtopping can occur at a lower elevation. This can lead to breach if the enough material is mobilized into a storm sewer under a levee that is parallel or adjacent to it. The Engineer Manual 1110-2-2902 Conduits, Pipes, and Culverts Associated with Dams and Levee Systems (USACE, 2020) recommends inspecting gravity pipes running adjacent to a levee embankment that are 15 feet horizontally of the levee toe or below a 1H:1V slope from that point (Figure 62). While the RA team is not aware of past issues related to a scenario like this in the area, it was determined it is not a risk driver. However, pipes must be inspected regularly to confirm that no defects or cracks are letting soil into them.

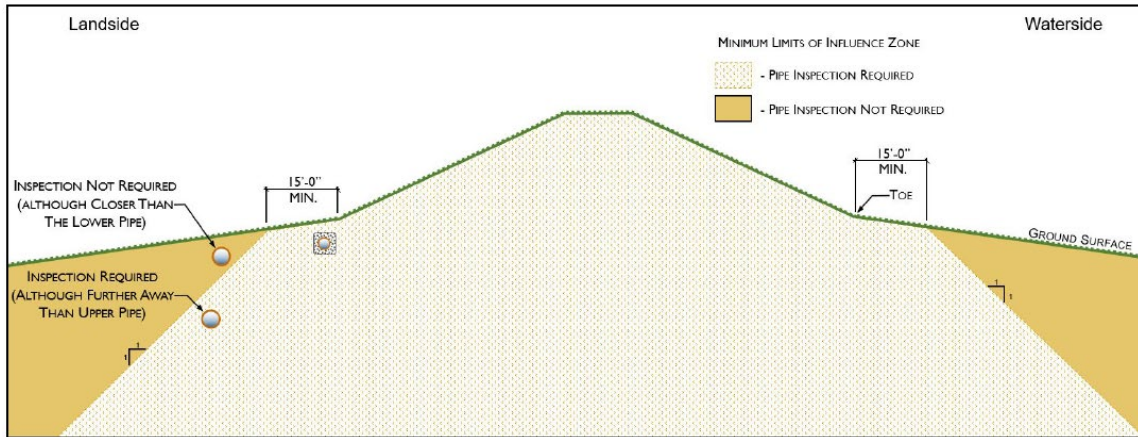


Figure 65. Required inspection zone for parallel pipes.

A.20. PFM 20: Scour from storm sewer discharge erodes the levee

If adequate end treatments and armoring are not provided for culvert or storm sewer outfalls, erosion could progress into the levee section. It is expected the design team would consider maximum erosion potential for possible outflows when designing these outfalls, and that risk to levee failure would be adequately reduced.

A.21. PFM 21: Tsunami overtops and breaches the levee

Tsunamis are a significant hazard of concern for the Pacific Coast, including the San Francisco area. However, the impact of a tsunami is lesser within the bay than it is further out along the directly impacted coastline. It is unlikely that a tsunami causes enough of a rise in water level along the project alignment to cause overtopping and it is extremely unlikely that a tsunami coincides with elevated water levels from a meteorological event. A tsunami would likely be associated with some amount of warning time and there would be much greater risks to the public in this area than the risk of a levee failure.

A.22. PFM 22: Contact erosion along foundation interface with rock dike

This PFM is related to soil contact erosion, which is an internal erosion process. This happens when fine particles in contact with a coarser soil move through the voids of the coarser layer. The water flow moves through the coarser layer and scours the fine materials. The migration of the fine material could lead to a change of volume and possible collapse. This type of contact erosion could be possible if fine material enters voids of the rock dike. The rock dike is comprised of quarry run rock, predominantly sandstone, with a wide range of grain sizes ranging from sand size to boulders. Figure 63 shows a cross section of the bulkhead wall on the rock dike. Due to the highly variable soils in the area, the rock dike could be in contact with any type of material. The rock dike is typically founded mostly on the Young Bay Mud, a very soft to firm estuarine clay and silt with minor sand layers. The rock dike is a known foundation condition that needs to be analyzed and addressed appropriately during design. The ground improvements, which include jet grouting or deep soil mixing, would mitigate if contact erosion could be

an issue. The rock dike was placed from 1878 to 1924 and there are no known issues related to contact erosion. It is not expected to change with the expected sea level rise. This PFM was not considered a risk driver.

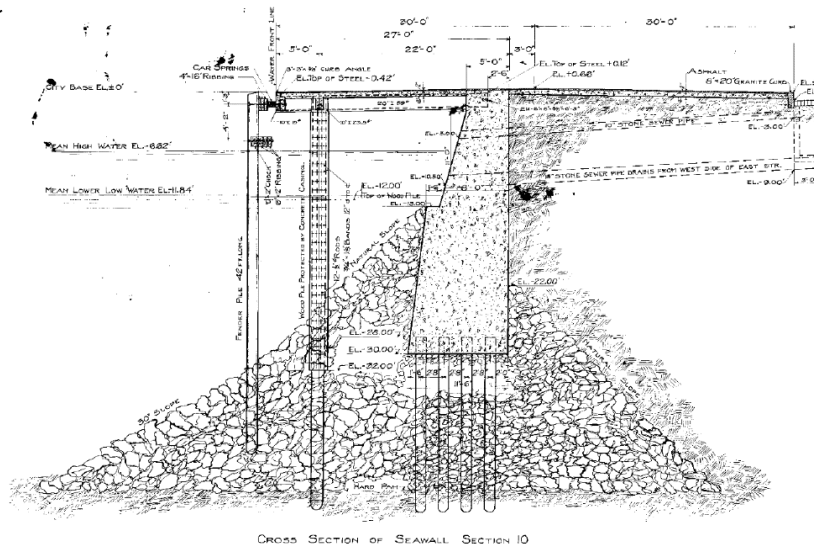


Figure 66. Cross section of bulkhead wall on rock dike.

A.23. PFM 23: Suffusion of soil behind or under the wall

Suffusion is a form of internal erosion which involves selective erosion of finer particles from the matrix of coarser particles of an internally unstable soil, in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. The migration of the fine material could lead to a change of volume and possible collapse. There have been no known issues associated with suffusion in this area. Similar to PFM 22, the ground improvements would mitigate if suffusion could be an issue. The team has determined that this PFM is not a risk driver.

A.24. PFM 24. Failure of a pier wall due to instability, structural failure, or overtopping failure

Numerous piers will be protected by floodwalls as part of the TSP (Figure 64). The walls will be constructed to 13.5' for the first action plan and will be at an elevation lower than the main line of protection for the second action. Depending on SLC the piers may need to be abandoned or the walls replaced. The pier side of the floodwalls is expected to be erosion resistant with either a splash pad or concrete pavement as part of the design, eliminating the chance of a typical overtopping failure by scour on the pier side. Vibrations induced by overtopping may still be a concern, and the design should account for them.

The walls will not be founded on soil and will be attached to concrete pier structures (Figure 65). This virtually eliminates the typical stability failure modes; the failure modes still exist but will likely not govern the design or probability of failure. Strength failures are unlikely as the design will follow current design standards, construction techniques, and modern QA/QC methods. The main driver of structural failure will most likely be the existing pier structure's condition. A new wall placed on a degraded structure will suffer from performance issues. Since these walls are assumed to be designed to the current state of practice, PFM 24 is not considered a risk driving failure mode.

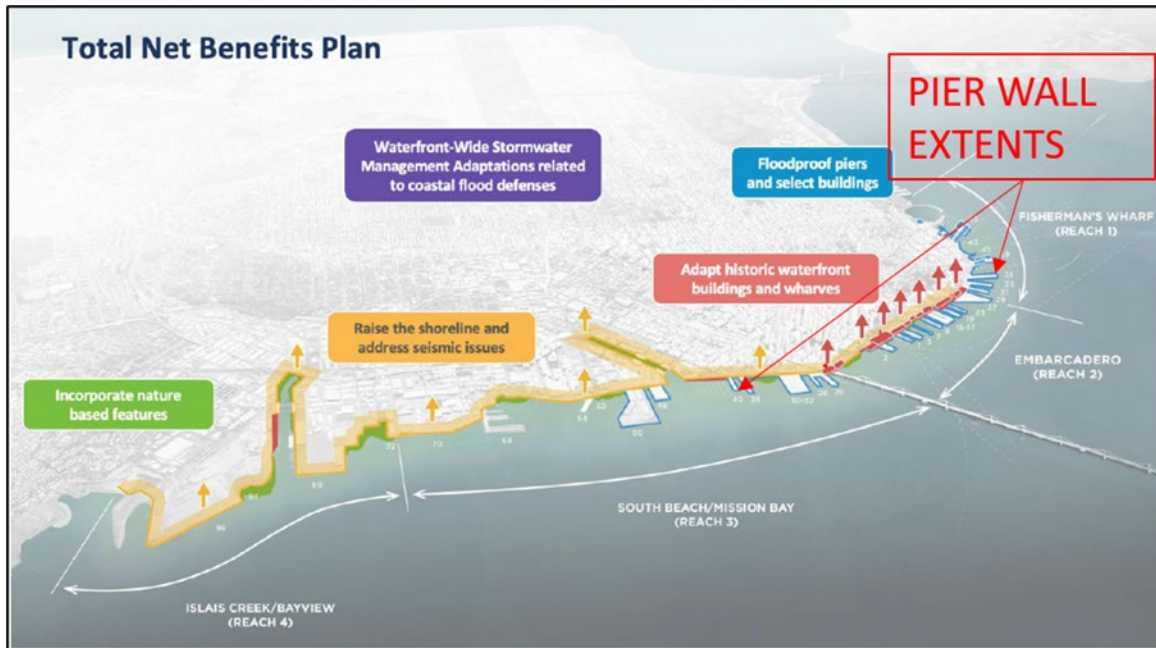


Figure 67. General project alignment from the Jan. 2024 Feasibility Report.

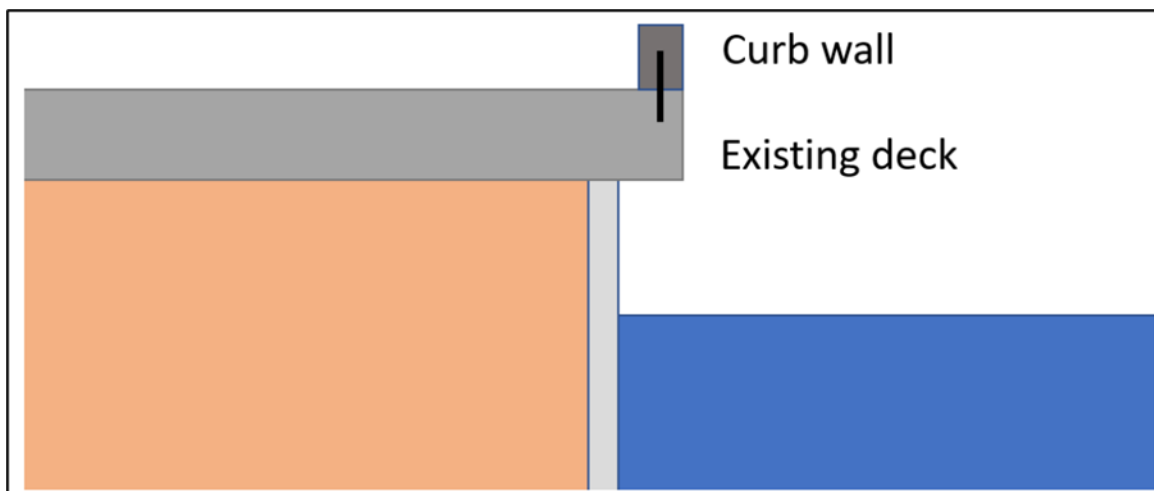


Figure 68. Curb wall example from the Jan. 2024 Feasibility Report Appendix B-3.3.4.

A.24.1. Recommendations.

Include inspections of the curb and pier in the O&M manual.

A.25. PFM 25: Failure of a new wharf structure due to instability, structural failure, or overtopping failure

The current TSP includes elevating of numerous structures and wharves and construction of a bulkhead wall as the line of protection. Conceptual wharf raises are shown in Figure 66 and Figure 67. The new wharf structures are expected to be designed to the current state of the practice resulting in minimal risk of failure for design load cases. The Ferry Building wharf raise appears to rely on existing pile foundations. The condition of those piles will likely drive the failure probability of a non-seismic wharf failure.

Should SLC exceed first action elevations, the wharves will require additional curb walls similar to Figure 65. If the curb wall is installed, the wharf pad will become an integral part of the line of protection. Any penetrations through the wharf pad then become water infiltration/leakage points. The wharf will also be subject to buoyant forces that may not have been accounted for during the original design of the piles. However, assuming due diligence will be completed during the design phase of the project, the piles are expected to perform as intended. Provided design considers the current state of the practice and all USACE design criteria, PFM 25 is not considered a risk driving failure mode.

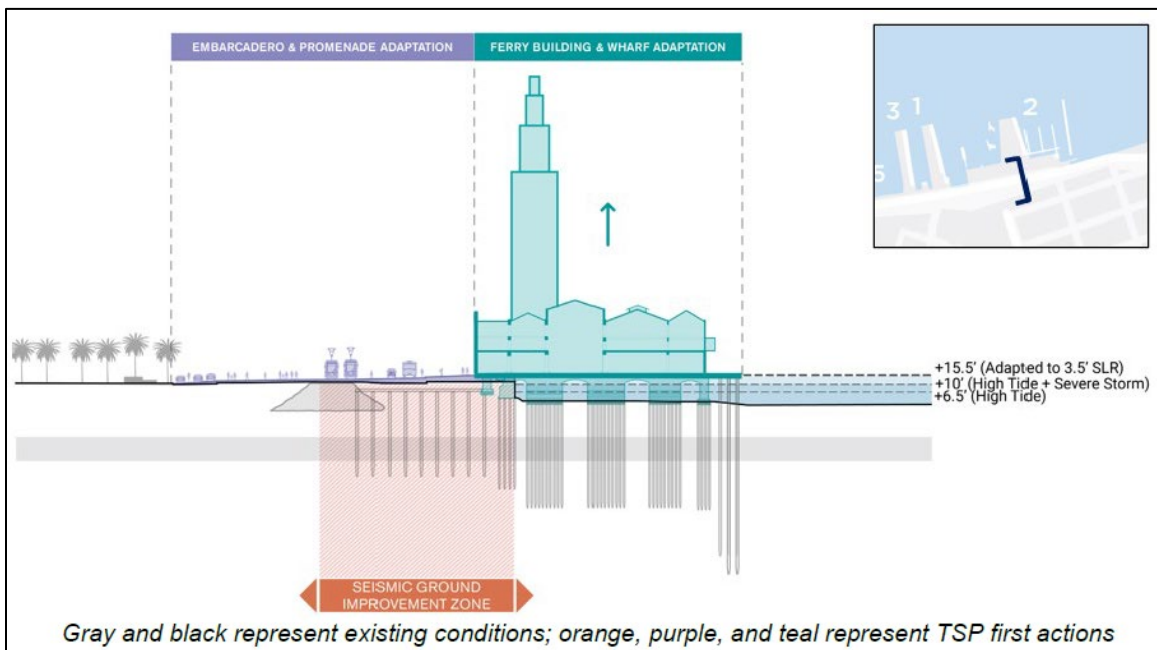


Figure 69. Conceptual wharf raise at the Ferry Building from Jan. 2024 Feasibility Report Section 5.1.1.

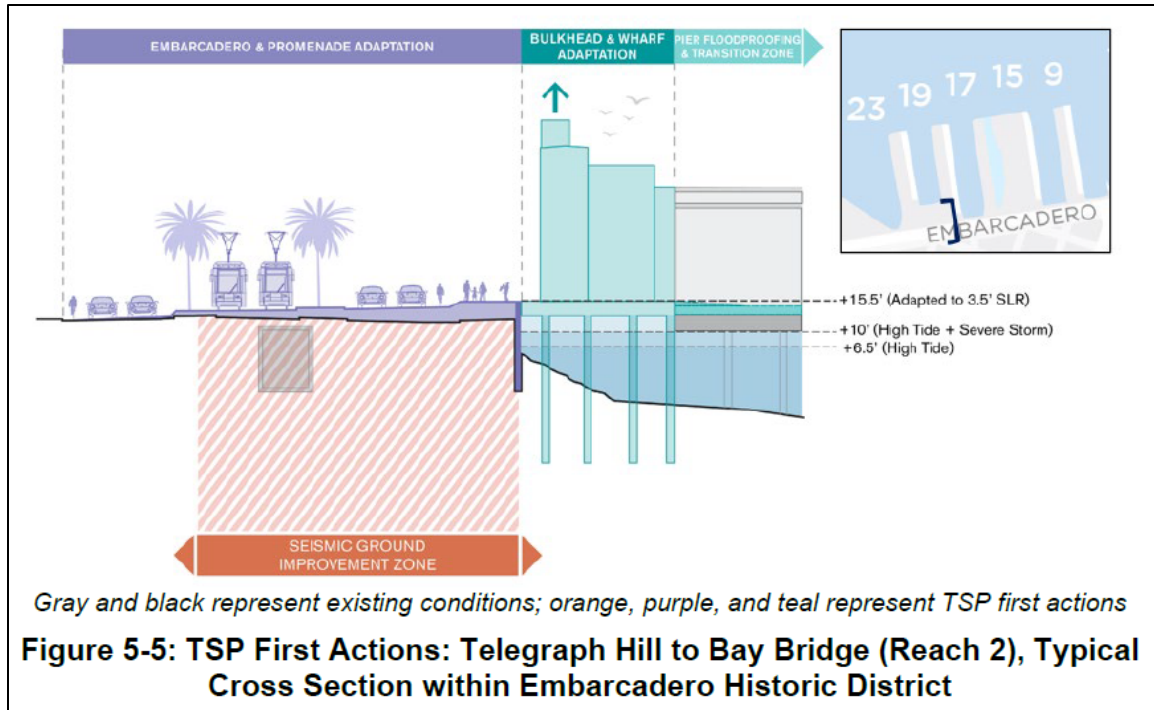


Figure 70. Conceptual wharf raise at the Embarcadero Historic District from Jan. 2024 Feasibility Report Section 5.1.1.

A.25.1. Recommendations.

Include inspection of the wharves in the O&M manual, to include the underside of the wharf and dive inspections of all submerged elements.

Consider the inclusion of uplift forces on the wharves during design.

A.26. PFM 26: Failure of an existing wharf structure due to instability, structural failure, or overtopping failure

The current TSP indicates that pier 80 wharves will not be replaced (Figure 68) but will be structurally tied to the new bulkhead walls. As can be seen in Figure 69, a large gantry crane is present on the wharf. Should that crane collapse and impact the wharf, a failure of the bulkhead wall is possible. The combined probability of the crane collapsing in such a way to fail the bulkhead wall should be considered remote. The combined probability of crane collapse and an extreme water event will result in a sufficiently low APF to support exclusion from risk driving consideration.

A more general failure of the wharves due to overloading from the shipping industry is possible. Due diligence during the design phase to evaluate the existing wharves is needed to ensure the bulkhead wall is not adversely affected by the wharves.

Since there is little information on the condition of the existing wharves at pier 80, this failure mode cannot be fully evaluated, however it is assumed that during the design phase the wharves will be evaluated and USACE guidance followed. As such, PFM 26 is not considered a risk driving failure mode.

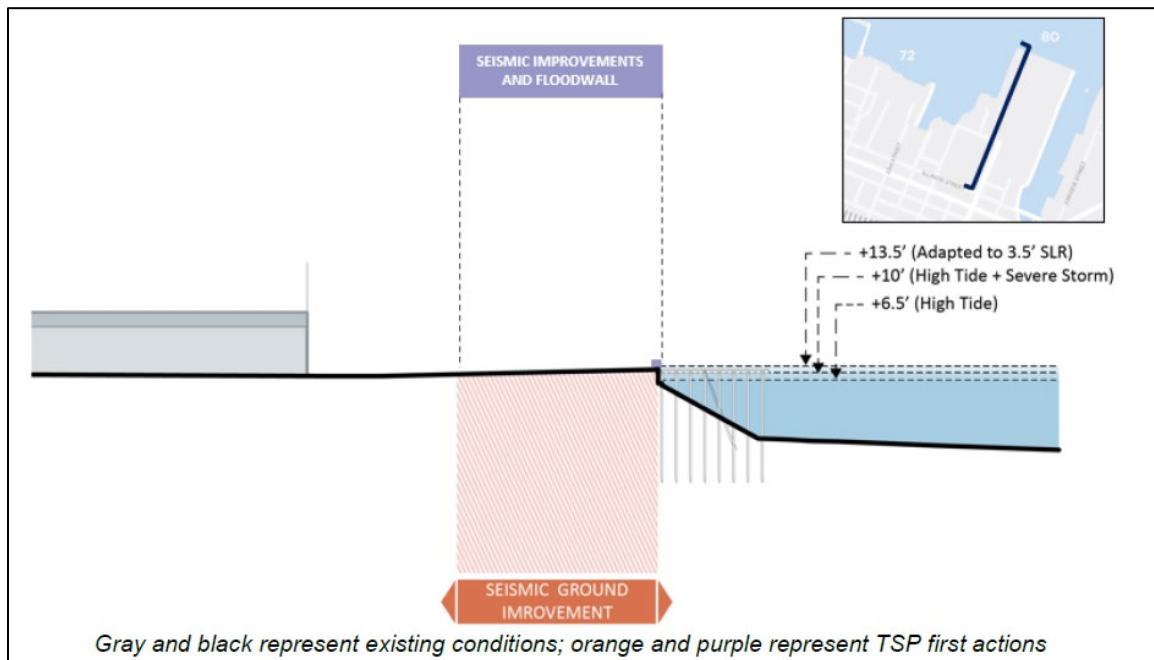


Figure 71. Conceptual first action at the Pier 80 from Jan. 2024 Feasibility Report Section 5.1.3.



Figure 72. Google Earth Image of Pier 80 from 01 April 2022 (Google).

A.26.1. Recommendations.

Evaluate the existing wharves to ensure they will not adversely affect the line of protection or isolate the line of protection from the wharves.

Include inspection of the wharves in the O&M manual if they are structurally tied to the bulkhead wall.

A.27. PFM 27: Internal migration into parallel storm sewer behind the wall

Similar to PFM 19 in levees, internal migration into a parallel storm sewer behind the bulkhead wall can occur. Soil particles can migrate into the pipe and a void at the interface between the pipe and the soil starts growing. As more material falls into a pipe along with the flow of water, the bulkhead wall could partially collapse. Overtopping can occur at a lower elevation and the system can breach. The Engineer Manual 1110-2-2902 Conduits, Pipes, and Culverts Associated with Dams and Levee Systems (USACE, 2020) recommends inspecting gravity pipes running parallel or adjacent to a floodwall that are within 8 feet horizontally of the floodwall or below a 1H:1V slope from that point (Figure 70). While the RA team is not aware of past issues related to a scenario like this in the area, it was determined it is not a risk

driver. However, pipes must be inspected regularly to confirm that no defects or cracks are letting soil into them.

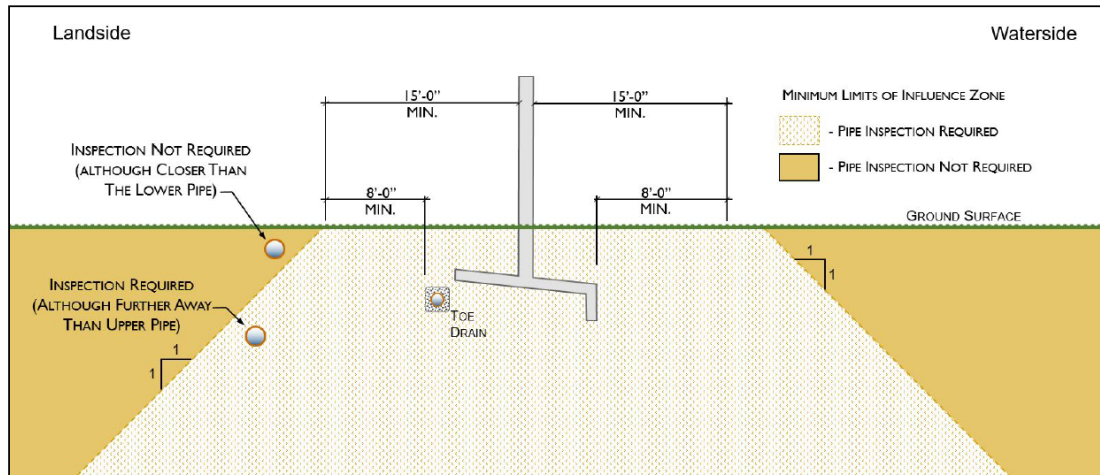


Figure 73. Required inspection zone for parallel pipes near floodwalls.

A.28. PFM 28: Global instability of bulkhead wall

This potential failure mode is regarding the global slope failure of the soil mass around the wall. These include rotational, translational or a combined failure of the soil mass. This type of failure can be assessed using limit equilibrium analysis methods within commercial software packages. Modeling of the floodwall with the appropriate soil strengths is performed to detect the slip surface with the lowest factor of safety, similar to slope failure modeling. Results of modelling of a global stability failure of the floodwall must meet the factor of safety criteria presented in the Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). This EM has criteria for T-Walls on shallow foundations, T-walls on deep foundations, I-Walls, etc.

Figure 71 shows an example of the circular slip surface below a wall and below piles. The soil shear strength parameters used for the modeling will be selected based on the laboratory testing program from the geotechnical exploration. Assuming that the soil strength values for the slope stability analyses will be appropriate for the soil surrounding the floodwall and that the design will meet the criteria, the team determined that this PFM is not a risk driver.

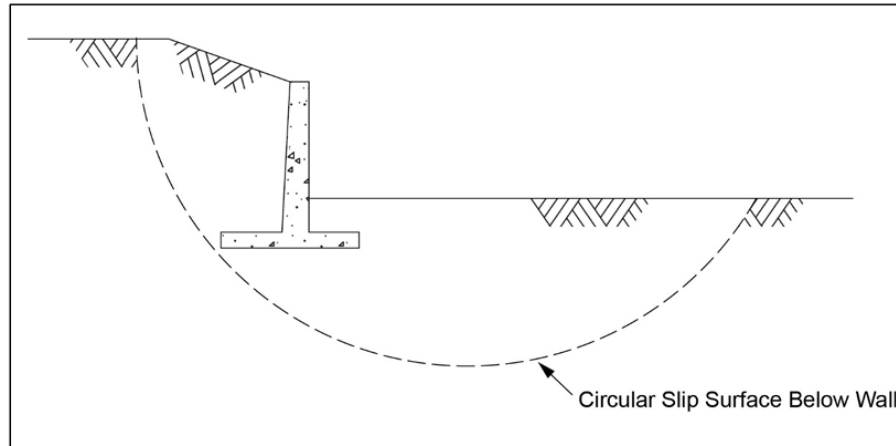


Figure 74. Example of slip surface relevant to global stability analysis under a floodwall.

A.29. PFM 29: Sliding, bearing, overturning failure of bulkhead wall

This potential failure mode is regarding the potential of breaching due to sliding, exceeding the bearing capacity or overturning of the bulkhead wall. These three failures are individual PFMs and may or may not happen at the same time. For sliding, the wall may fail due to excessive horizontal deformations. The potential for a sliding failure can be assessed by comparing the resisting shear forces against the acting applied shear forces on the wall. For bearing capacity, the wall may fail when the pressures that can be permitted against the foundation soils exceed the allowable bearing capacity. The bearing capacity will depend on the loading conditions, compressibility of the soil and geometric considerations. Lastly, overturning of the wall may happen. However, the wall will always fail first due to sliding, exceeding bearing capacity or structural damage before the structure overturns.

Assessment of these potential failure modes must follow the equations and criteria presented in the Engineer Manual 1110-2-2100 Stability Analysis of Concrete Structures (USACE, 2005) and Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). The team determined that this PFM is not a risk driver if the design uses the corresponding analyses and meets the criteria.

A.30. PFM 30: Prop wash erosion undermines bulkhead wall

The bulkhead wall design will include the rock dike and any additional armoring that is required to mitigate erosion. The primary driver of armoring size will likely be wind-wave estimates, however, the design team should ensure the controlling erosive conditions are mitigated for, including wind-wave, vessel wake, under water turbulent jets (i.e. prop wash), and flow velocities. If armoring is under-designed compared to future conditions for prop wash, routine inspections and maintenance should address this issue prior to a full failure of the bulkhead wall.

A.31. PFM 31: Seismic event leads to a global instability of the wall and soil

Exclusion discussed in A.1.

A.32. PFM 32: Structural failure of a bulkhead wall

Numerous bulkhead walls will be used on the project; see Figure 66, Figure 67, Figure 68 for conceptual bulkhead wall configurations. The current level of design does not provide sufficient detail to fully evaluate structural failure of a bulkhead wall. Assuming the design incorporates current state of the practice methodologies and USACE design standards, it is unlikely a strength failure under static conditions will develop. Providing adequate concrete cover and proper mix design are key to hydraulic concrete longevity. Concrete cracking from any source must be treated to minimize saltwater intrusion. Therefore, a robust O&M plan must be implemented. Assuming due diligence during design, structural failure of a bulkhead wall is not considered a risk driving failure mode.

A.32.1. Recommendations.

Include crack width limits and remediation measures in the O&M manual.

Include post-earthquake inspections in the O&M manual.

Update the TSP to include current USACE design criteria (i.e., walls are state to be designed to EM 1110-2-2502 (1989), the current version is from 2022).

A.33. PFM 33. Ship impact leads to failure of a bulkhead wall

The Port of San Francisco (POSF) is an active and busy port hosting ships of various sizes (Figure 72). The cruise ships that dock at Pier 27 (Figure 73) was the largest observed ship to utilize the port. However, large container ships on-route to the Port of Oakland pass in close proximity to the POSF (Figure 74). It is assumed that some level of impact load will be accounted for in the design of the wharves and bulkhead walls. With that assumption, only the larger cruise ships or cargo ships are likely to cause significant damage to the bulkhead walls. Representatives from the POSF indicated that siltation within the mooring areas is an ongoing issue; it is unclear if there is enough water depth near the bulkhead walls to have one for the larger ships impact it at speed, or if the ship would run aground first.

For a large container ship to impact the bulkhead walls it would need to make a hard turn toward the port during transit. This could be the result of a loss of power, similar to the Dali (Port of Baltimore, 2024), or from human error. It is likely that an out-of-control cargo ship would both impact wharves and docked ships prior to hitting a bulkhead wall and would likely strike the bulkhead wall at an oblique angle. Both scenarios will reduce the impact force on the bulkhead walls. When considering all factors, impact of a cargo ship remote and design for a direct impact would be cost prohibitive.

The cruise ship that docks at Pier 27 is the most likely ship to impact a bulkhead wall resulting in failure of the wall. The cruise ship is nearly as large as the cargo ships that transit the area, and docks at the pier frequently. Docking is generally done at low speed with the assistance of a tug, leaving electrical failure, mechanical failure, and/or human error as the primary cause of ship impact. There is the potential that the cruise ship runs aground prior to impacting the bulkhead wall. Ship impacts are not discussed within the

Feasibility report and design loads have not been determined, but considering the size of cruise ship it is unlikely that the design will account for an impact at speed. Intervention is possible due to the limited damage that would occur, measuring on Google earth indicates that the docked cruise ship is approximately 120' wide. A breach of approximately 200' is a reasonable assumption and within a length that can be addressed quickly to prevent flooding.

Due to the high probability of intervention and remote likelihood of a ship impact, PFM 33 is not considered a risk driving failure mode.

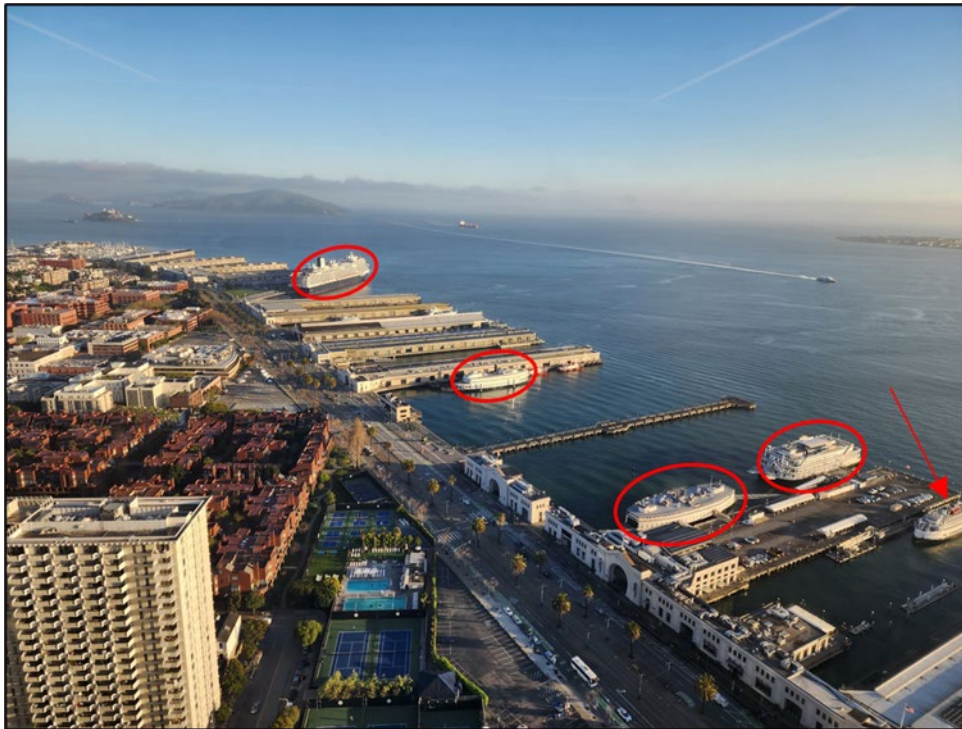


Figure 75. Docked ships at the Port of San Francisco (USACE).

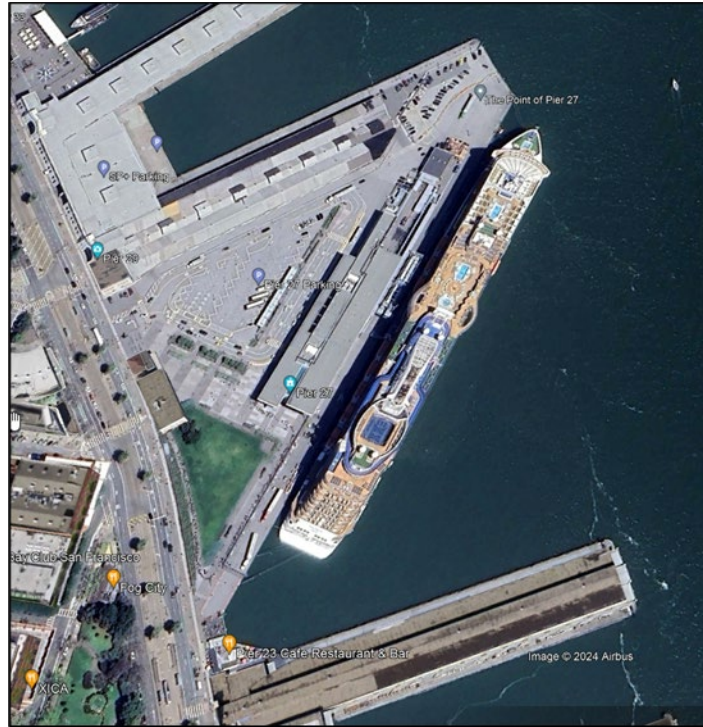


Figure 76. Cruise ship docked at Pier 27 from 22 April 2023 (Google).



Figure 77. Cargo ship passing near the Ferry Building (USACE).

A.33.1. Recommendations.

Develop risk informed impact loads for the bulkhead walls. Consider installing dauphins or other impact protection where feasible. Include language specific to ship impacts within the EAP and have materials stockpiled or contracts for procurement of equipment and materials to intervene.

A.34. PFM 34. Surcharge loading fails a bulkhead wall

The POSF is a highly active commercial area with numerous structures will be within the project right of way (Figure 64). Given the nature of the area, it can be assumed with a high level of confidence that at multiple points within the project life surcharge loads will be placed on the bulkhead walls. Assuming the POSF has a permitting process as part of their existing O&M plan, it is unlikely that loads in excess of design will be permitted. If permitting is not part of the existing O&M plan, it should be included in the levee system O&M manual. This permitting would be in addition to Section 408 permits for levee system alterations.

The design phase is assumed to include a nominal surcharge load. Until the load has substantially exceeded that surcharge loading failure probability is remote. The current level of design is not sufficient to fully evaluate this failure mode, but it is assumed that the design level surcharge loads will be reasonable for the anticipated surcharge loads and the permitting process will provide a reasonable accounting of loads for evaluation by an engineer. For this reason, PFM 34 is not considered a risk driving failure mode.

A.34.1. Recommendation.

Determine what the current POSF permitting process is and whether it fully addresses surcharge loading. If the process does not sufficiently address surcharge loading, add language to the levee system O&M manual to establish a permitting process within the POSF that will adequately address surcharge loading. This permitting process would be in addition to the Section 408 process since there may be instances where a Section 408 permit is not required but a surcharge is present.

A.35. PFM 35: Tree within VFZ topples and damages wall

This PFM is regarding a tree toppling and damaging any of the walls. If there is significant damage to the wall and a high water event happens before it can be repaired, water could push and cause more damage that could lead to breach of the flood protection. As mentioned in PFM 15, there are intentions of incorporating vegetation in this project. This could mean that trees could be planted close to the walls. The USACE policy has been to keep floodwalls “vegetation free”. This means that there is a vegetation free zone of 15 feet away from the wall (minimum) on the land side and water side. In addition, there is an 8 feet horizontal clearance from the wall foundation. Lastly, there is a vertical clearance of 8 feet above the top of the floodwalls.

While the USACE has recently adopted risk-informed design process that could allow vegetation other than grass on levees, floodwalls are different. Best practice is to retain the aforementioned minimum distances, horizontal and vertical, in which there should be no trees or woody vegetation. Only grass or other small herbaceous vegetation could be allowed as long as the roots do not cause damage to the walls. This PFM was determined to be a non-risk driver if trees are planted away from the vegetation free zones.

A.36. PFM 36: Overtopping erosion behind the bulkhead wall

The bulkhead wall is adjacent to The Embarcadero, so any overflow from storm tides or wave overtopping from wind-waves are unlikely to initiate erosion along the paved surface of the roadway. Minor amounts of overflow or wave overtopping are likely to cause nuisance flooding and make the roadway and sidewalks unsafe for vehicles and pedestrians. If the capacity of the bulkhead is being exceeded with overtopping/overflow, the critical project features are likely to be the levees and floodwalls that have much smaller volumes of landside material to erode and are less likely to have paved features that help to armor from erosion.

A.37. PFM 37: Wave impact failure of a bulkhead wall

The current TSP assumes a uniform 2' wave height across the project. Assuming the design will account for additional wave heights as needed during design, the probability of failure is likely very low. Additionally, the bulkhead walls generally act as retaining walls, so bayside loads will reduce the overall load in the system.

Assuming the design incorporates the recommendations from PFM 33, wave impact loading is not expected to exceed ship impact loading. For this reason, PFM 37 is not considered a risk driving failure mode.

A.38. PFM 38: CLE along penetrations through bulkhead wall

This PFM requires a crack or void around a pipe or culvert that would allow for a concentrated leak erosion. Similar to PFM 4, CLE through a crack along a pipe (drainage, water supply, sewer, gas, etc.) can lead to breach. The pipes could either be in the foundation under the bulkhead wall or through the wall itself. For this PFM to occur, there needs to be sufficient flow and the hydraulic shear stress should exceed the material critical stress to start movement of particles. As soil particles detach, the crack becomes larger and can lead to failure of the structure.

Existing pipes and other pipes that could be necessary to be added need to be considered in the design. Waterstops should be included around penetrations that go through the walls to avoid flow or migration of water. All pipes need to be inspected and monitored to detect issues. No distress or failure has been observed existing penetrations that are linked to CLE. Assuming inspection and monitoring efforts to prevent issues related to CLE along penetrations, this PFM was determined to be a non-risk driver.

A.39. PFM 39. Collapse of the BART tunnel fails a bulkhead wall

The BART tunnel crosses the levee alignment near the Ferry Building (Figure 75). Except for very specific circumstances, which are detailed below, the failure, subsequent inundation, and consequences resulting from the collapse of the BART tunnel is not associated with the levee system. That risk would be fully associated with the BART system. The risk would only shift to the bulkhead wall and levee system if the wall or piers are designed to rely on the BART tunnel system for stability and the failure of the wall is the driver of inundation of the interior. Another way the risk shifts from the BART system to

the levee system is if the feasibility study—with coordination from all stakeholders—determines that it is in the federal interest to incorporate FRM features into the BART system, making the BART system FRM features part of the levee system.

Should the tunnel fail under the floodwall or bayside of the floodwall, a collapse would certainly begin to inundate the tunnel and fill the interior of the leveed area eventually equalizing to the elevation of the bay. Flooding would also impact the city of Oakland and any other connected areas. The levee system is likely to remain intact provided the design does not rely on the BART tunnel for stability. Scour of the bay mud is possible and may eventually cause an instability of the bulkhead wall, but intervention to limit flow is equally possible.

Since this failure mode and resulting risk are primarily associated with the BART system and not the bulkhead wall, this PFM is not considered a risk driving failure mode.

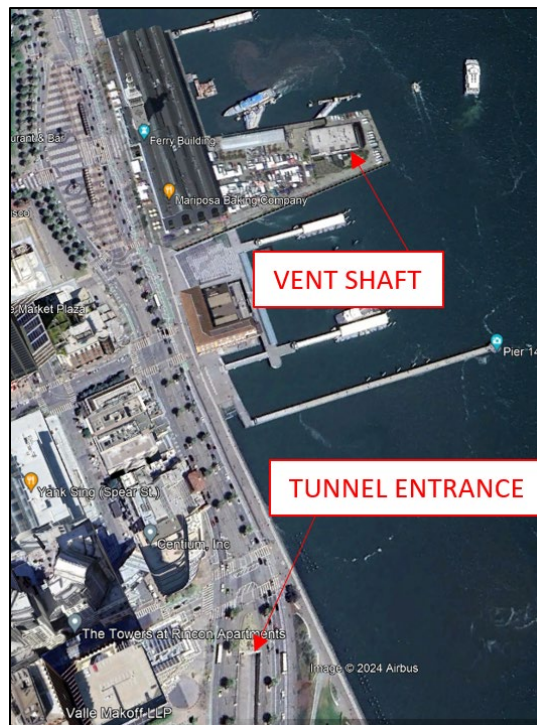


Figure 78. BART Tunnel near the Ferry Building (Google).

A.39.1. Recommendations:

Design of the bulkhead wall should not rely on the BART tunnel being intact or in place for stability.

A.40. PFM 40. Construction of a new BART tunnel leads to failure of the bulkhead wall

According to the POSF, plans have been developed by BART for a future BART tunnel expansion in the area between the Ferry Building and the Bay Bridge. These plans are not available for this risk assessment. Should the levee system be constructed prior to the construction of the BART tunnel

expansion, a Section 408 permit will be required. As part of that permitting process it must be proven that an expansion will not adversely affect the line of protection and that temporary emergency flood protection measures have been designed/established in the event of a failure. If the tunnel is constructed prior to the construction of the levee system, this PFM is no longer applicable; PFM 39 would need to be modified to incorporate the new tunnel.

Due to the Section 408 permitting requirements, PFM 40 is not considered a risk driving failure mode.

A.40.1. Recommendations.

Utilizing the risk communications process, established POSF and USACE protocols, obtain a memorandum of agreement or a memorandum of understanding with BART that outlines the Section 408 process for modifications and specifically addresses modification or expansion of BART tunnels that cross the line of protection.

A.41. PFM 41: Settlement of the bulkhead wall and fill leads to more frequent overtopping

This PFM alludes to settlement in some areas along the bulkhead wall that can result in lowering the top of the wall below the design elevation and more frequent overtopping. Vertical deformation when settlement is not accounted for can contribute to more recurrent overtopping that could lead to breach of the structure. Chapter 7 discusses overtopping scenarios and associated consequences due to breach. The two main contributors to vertical subsidence of a levee are due to seismic events and traditional geotechnical consolidation settlement. This PFM only involves the traditional settlement. The soils on where bulkhead wall is are mostly fill, Young Bay Mud (soft to firm clay) and Upper Layered Sediments (layered sequence of sands clays and gravels). The current seawall was experienced settlement in the past, but the stresses added by the additional weight of new construction would likely cause additional settlement that needs to be take into account. The ground improvement will stiffen the matrix of the foundation soils and reduce settlement. A plan to monitor the settlement is necessary whether ground improvement is incorporated or not to detect if settlement is more than expected. The team has determined that this PFM is not a risk-driver.

A.42. PFM 42: Seismic failure of external infrastructure and buildings

Exclusion discussed in A.1.

A.43. PFM 43: Failure of the wharf leads to bulkhead wall failure

A catastrophic failure of the wharf would require multiple unlikely actions to occur. The initiating mechanism that would fail the wharf would need to be an earthquake or ship impact to such a degree that the entire wharf pulled away and compromised the bulkhead. While an earthquake, ship impact, or other catastrophic event could cause some damage to the wharf, it is extremely unlikely that it would progress to a degree that would damage and fail the bulkhead.

A.44. PFM 44: Failure of dewatering measures leads to construction personnel risk

The team acknowledged the life safety risk implications of the complex construction of miles of project features that will require some amount of work for personnel in a dewatered condition below the water surface. If dewatering measures are not properly designed or the risk of cofferdam overtopping is not adequately assessed, flooding of a dewatered area could result in life safety risk during construction. The design team should ensure that a robust dewatering plan is required during construction to account for not only budget and schedule risks, but also life safety risks.

A.45. PFM 45: Overdredging leads to global stability of bulkhead wall

The bulkhead wall design will include the rock dike and any additional armoring that is required to mitigate erosion. The primary driver of armoring size will likely be wind-wave estimates, however, the design team should ensure the controlling erosive conditions are mitigated for, including wind-wave, vessel wake, under water turbulent jets (i.e. prop wash), and flow velocities. If armoring is under-designed compared to future conditions for prop wash, routine inspections and maintenance should address this issue prior to a full failure of the bulkhead wall.

A.46. PFM 46: Fire at a pier compromises a bulkhead wall

Should a fire at a pier occur, the bulkhead walls are unlikely to be affected. Concrete is fire resistant with the most probable damage from a low heat fire being cracking and spalling. Should a fire heat the concrete to greater than 500 °F other damage types may occur. Damage is likely limited to the top portion of the wall and is unlikely to affect the overall strength or stability. Repairs and/or temporary flood protection measures (intervention) may be needed to prevent premature overtopping. Intervention is considered probable due to localized nature of the fire and damage. Due to the low combined probability of a fire and high water event, limited damage to the line of protection, and probable intervention, PFM 46 is not considered a risk driving failure mode.

A.47. PFM 47: Settlement-induced bending moment failure

Pile-founded T-walls may be constructed as part of this project. If constructing battered piles as part of a T-wall, care needs to be taken to avoid any settlement-induced bending moments that can be applied to the piles in conditions where fill is being placed over compressible foundation material. When this fill causes foundation settlement, a bending moment is exerted on the pile that reduces the ultimate capacity of the pile under flood loading. Given the USACE experience with this PFM in the design of New Orleans area floodwalls, this issue should be accounted for in design and review. Additionally, the height of these T-walls is likely to be lower than in New Orleans, so the design may find no need for battered piles or floodwalls on top of placed fill material with compressible soils.

A.48. PFM 48: Overtopping erosion behind surface floodwall

Still water overflow and wave overtopping can occur when the capacity of the system is exceeded. If landside erosion is significant, the scour hole could destabilize the wall under flood loading. The proposed floodwalls for the project are pile-founded T-walls, which have both a base slab that may limit some erosion and also remain stable in conditions with some amount of scour hole developed. There have been no known T-wall failures from overtopping in modern USACE history. For any type of wall, potential scour from overtopping should be assessed and either mitigated through scour pads or accounted for in stability modeling.

A.49. PFM 49: Sliding, bearing, overturning failure of surface floodwall

This potential failure mode is regarding the potential of breaching due to sliding, exceeding the bearing capacity or overturning of the new floodwalls. The design of these floodwalls is either I-walls (cantilever sheetpile), T-walls or L-walls. The three failures are individual PFMs and may or may not happen at the same time. For sliding, the wall may fail due to excessive horizontal deformations. The potential for a sliding failure can be assessed by comparing the resisting shear forces against the acting applied shear forces on the wall. For bearing capacity, the wall may fail when the pressures that can be permitted against the foundation soils exceed the allowable bearing capacity. The bearing capacity will depend on the loading conditions, compressibility of the soil and geometric considerations. Lastly, overturning of the wall may happen. However, the wall will always fail first due to sliding, exceeding bearing capacity or structural damage before the structure overturns.

Assessment of these potential failure modes must follow the equations and criteria presented in the Engineer Manual 1110-2-2100 Stability Analysis of Concrete Structures (USACE, 2005) and Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). The team determined that this PFM is not a risk driver if the designs use the corresponding analyses and meet the criteria.

A.50. PFM 50: Ship impact leads to failure of a floodwall (T-wall, L-Wall, or I-Wall)

PFM 50 is similar to PFM 33; for additional information see PFM 33. PFM 50 is less likely to occur than PFM 33 due to the proximity of ships to the wall and the potential to run aground prior to reaching the wall. As such, PFM 50 is not considered a risk driving failure mode.

A.51. PFM 51: Wave impact leads to failure of a surface floodwall (T-Wall, L-Wall, or I-Wall)

PFM 51 is similar to PFM 37 and additional information can be found there. The biggest risk to surface floodwalls from waves is due to the potential to undermine the bayside foundation material. The walls are expected to meet USACE guidance and consider wave action. Provided due diligence is practiced during

design and proper QA/QC is practiced during construction, PFM 51 is not considered a risk driving failure mode.

A.52. PFM 52: Seismic event leads to global stability or structural failure of floodwall

Exclusion discussed in A.1.

A.53. PFM 53: BEP through foundation under surface floodwall

This PFM is regarding backward erosion piping (BEP) through the foundation material underneath the floodwalls (T-walls or L-walls). As the sea water surface elevation rises, seepage occurs in a continuous (waterside to landside) zone of uniform sand beneath the wall base. An unfiltered exit must exist at the landside toe or at some distance away from it on the landside. Sufficient seepage forces detach soil particles in the uniform sand layer at the seepage exit, initiating backward erosion piping. The base of the T-walls and I-walls can act as a roof. Sufficient horizontal gradients exist for progression of a pipe towards the sea. If detection and intervention are unsuccessful, BEP continues, and instability can collapse the floodwall. Water overtops the structure with uncontrolled release and breaches the system. These walls will be constructed using deep foundations using H-piles or pipe piles. The existing fill where the walls will be constructed consists of heterogeneous sandy fill that includes dune sand and debris. This material or any other poorly-graded fill could erode due to BEP. T-walls and I-walls will incorporate a steel sheet pile cutoff, extending 20-30 feet vertically beneath the wall base, to reduce underseepage and uplift pressures on the structure. The design must follow the Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). In addition, ground improvements could limit erosion potential. The team determined this PFM is not a risk driver.

A.54. PFM 54: Unauthorized modification to surface floodwall leads to ineffectiveness

Any modifications to the project features and associated right-of-way would need to go through the 408-permitting process. However, many times modifications are made without proper review and approval and go unnoticed and uncorrected during the routine inspection process. It is unlikely that a modification could be major enough to severely affect the performance of a floodwall without being identified during the next inspection and corrective action being taken. Minor modifications (e.g., small encroachments) are unlikely to impair project performance. This PFM, though, highlights the importance of a thorough inspection program throughout the life of the project.

A.55. PFM 55: Vehicle impact leads to failure of a surface floodwall (T-Wall, L-Wall, or I-Wall)

Low speed vehicle impacts pose little risk to the floodwalls and can be managed through the typical design process. Commercial vehicles, being larger and heavier, pose a risk at moderate to high speeds. Damage could vary from minor surface cracking/spalling to complete collapse of a floodwall monolith. Generally speaking, the opportunity for high-speed impacts is low due to the project alignment, traffic, and speed limits. Moderate speed impacts are likely to produce surface damage and are unlikely to neutral

to produce a monolith collapse. Inspection and intervention post-impact will be possible and should be considered likely. Due to the limited speed of an impact and low probability that a large enough vehicle traveling at moderate speeds impacts the walls, PFM 55 is not considered a risk driving failure mode.

A.55.1. Recommendations.

In areas of high traffic or areas with heavy commercial vehicle traffic, consider protection of the floodwalls using impact protection approved by the authority having jurisdiction.

A.56. PFM 56: Structural failure of floodwall reinforcement

This failure mode is similar to PFM 32 but is considered less likely. For additional information see PFM 32. Surface floodwalls, are not typically exposed to saltwater from the bay unlike the bulkhead walls (seawalls), leading to less degradation of the walls over time. Since PFM 56 is less likely than PFM 32, PFM 56 is not considered a risk driving failure mode.

A.57. PFM 57: CLE along interface between surface floodwall and soil

This PFM assumes that CLE happens at a crack, void, or defect between the soil and the floodwall. When constructing the floodwall, it could be possible that the floodwalls will not have direct contact with the soil. This could lead to a preferential path for flow. The soil could start eroding during a high sea event. The floodwall could collapse, leading to breach of the flood protection system. Similar to PFM 56, the cutoff wall and ground improvement will limit erosion at the soil-wall interface. The team has determined that CLE between the soil and floodwall is not a risk driver.

A.58. PFM 58: Animal burrows leads to shortened seepage path for BEP or CLE under a surface floodwall

This PFM is regarding a failure of the floodwall due to concentrated leak erosion (CLE) or backward erosion piping (BEP) that initiates in a single or a series of animal burrows. It is assumed for this PFM that there is continuous erosion starting in an animal burrow under the floodwall as sea water level rises leading to breach. The most common burrowing rodents that could be found in the project area are gophers, but they are not common. There are no previous incidents of animal burrows in this area. Currently, there is no operation and maintenance plan in place to control animal activity. In addition, the floodwalls will be inspected properly (frequency to be determined) and animal activity can be detected during these inspections. Furthermore, floodwall maintenance personnel or residents could also notice if extensive burrowing is occurring near the flood control structures. It is assumed that the floodwalls will be well maintained. Mitigation measures can be implemented if necessary if erosion is noticed due to animal activity. The scenario of animal burrows under floodwalls in the project area could even be less likely than burrows in the levees (discussed in PFM 13). Soil improvement under the floodwalls could also make it less likely for animals to be able to dig through. Even if the unlikely scenario of animals

digging under the floodwall happens, this event requires many nodes in an event tree that would lead from initiation to breach of the levee. Due to all these reasons, the team concluded that this PFM is not a risk driver.

A.59. PFM 59: Fire at a pier compromises a surface floodwall

This PFM is similar to PFM 46; see PFM 46 for additional information. Damage to a surface floodwall will likely be greater than a bulkhead due to the amount of exposed surface area. The damage is expected to be surficial for nearly all fires, but the possibility exists that a wall could be damaged to an extent that failure during a subsequent flood event is possible. The probability of a fire that is both large enough and hot enough to compromise a significant portion of floodwall is assumed to be low, therefore the joint probability of the fire and flood event is very low. Intervention is likely due to the localized effects of a fire. Due to the assumed low probability of the joint event and probable intervention, PFM 59 is not considered a risk driving failure mode.

A.60. PFM 60: Operational failure to install a closure

Failure to install a closure, historically, has approximately a 10% chance of occurring. Due to the limited number of closures on the system—currently 6 are shown in the TSP but others will be needed—and ample available POSF staff (>100 personnel), it can be assumed the likelihood of failure to install is one to two orders of magnitude lower. The design has not progressed sufficiently to evaluate the system of closures in detail; the total number of closures has not been determined and the location of all closures has not been determined.

Assuming the POSF has adequate and trained staff, operational and available equipment, and is involved in the planning of installation procedures this failure mode is unlikely to occur. For situations where stoplog closures are used, partial height installations for low elevation flood events may reduce required resource needed by the sponsor both before and after the flood event. For these reasons, PFM 60 is not considered a risk driving failure mode.

A.60.1. Recommendations:

When considering closure types, follow the hierarchy shown below.

1. Elimination of the closure or provide a ramp up and over the levee.
2. Moveable gates (i.e., roller gates, swing gates, trolley gates).
3. Structural assembled closures (i.e., stoplogs, bulkheads, truss/frame closures).
4. Passive closure systems (i.e., bottom hinge, vertical).
5. Earthen assembled closures (i.e., sandbag, soil pile and plastic sheet, soil baskets).

Once the design has progress and closure types/locations have been determined. Evaluate the full system of closures to determine the weakest link. This process must consider flood prediction, EOC standup trigger elevations, staff proximity to the site (are their ingress routes flooded/unpassable), the assumed staffing and number of teams, installation trigger elevations, installation order, travel time between closures, installation time, missing parts or equipment, weather, etc. and must be compared to project hydrographs to determine a probability of failure to install.

A.61. PFM 61: Closure failure due to train impact

The TSP indicates that there are 4 closures that will include train tracks, two light rail and two freight lines. The city owns the light rail lines, and the POSF owns the freight lines making coordination for closures a much simpler process. Bridges where the closures will be placed are moveable and have signaling on the tracks currently that could be modified to indicate when a closure is in place. Trains in the area are generally traveling at low speeds, allowing for relatively quick stops. Should the signaling system fail and the train not stop failure of the closure is certain, however the joint probability is assumed to be very low. For these reasons PFM 61 is not considered a risk driving failure mode.

A.62. PFM 62: Closure failure due to vehicle impact

Current federally authorized levees have had no known closure failures due to vehicle impacts. Typically, roads are closed with signage. Barriers can be placed to prevent vehicles from accessing the area. Access controls and signage should be placed in accordance with the AHJ and are believed to be an acceptable level of protection from vehicle impact. For these reasons PFM 62 is not considered a risk driving failure mode.

A.63. PFM 63: Structural failure of a closure

Current federally authorized levees have had no known structural closure failures. The most likely failure mechanism for a structural failure in a new levee system is a construction or fabrication defect; long term reliability is governed by material degradation. If the contractor performs adequate QC and the USACE performs adequate QA, defects are unlikely to be substantial enough to cause a structural failure of the closure. Long term degradation can be mitigated through material selection and adequate maintenance. Provided the current state of the practice and USACE guidance is used, PFM 63 is not considered a risk driving failure mode.

A.64. PFM 64: Closure leakage leads to erosion which fails the adjacent wall

Leakage can occur at closure gates under flood loading either due to worn out seals, improper operation, or other mechanical issues that prevent a tight closure from being made. This leakage can occur along the sill of the gate or along a vertical face. If leakage volumes are large enough and directed at erodible

material, erosion could undermine the adjacent walls and cause failure. Even if some leakage occurs, this PFM is unlikely to initiate significant erosion due to the low volume and low energy associated with leakage. If erosion initiated, the short duration of tide events would limit the extent of erosion. With proper inspection and maintenance, leakage should not occur in any substantial way and can be monitored during a flood event, with placement of material and sandbags helping to mitigate any leakage concerns, if needed.

A.65. PFM 65: Seismic event occurs and fails closure sill

Exclusion discussed in A.1.

A.66. PFM 66: Debris impact leads to closure failure

There is a potential for debris to impact a closure due to loose vessels, dilapidated piers, or other miscellaneous debris (Figure 76). There is little fetch perpendicular to the closures for waves to push debris toward a closure. Additionally, the prevailing wind direction will tend to take debris into the bay rather than into the creeks. A nominal debris loading should be included in the design but is unlikely to account for a large vessel or pier that breaks away. Debris is likely to become trapped on the bridge or guardrails prior to impacting the closures (Figure 77). For these reasons, PFM 66 is not considered a risk driver.

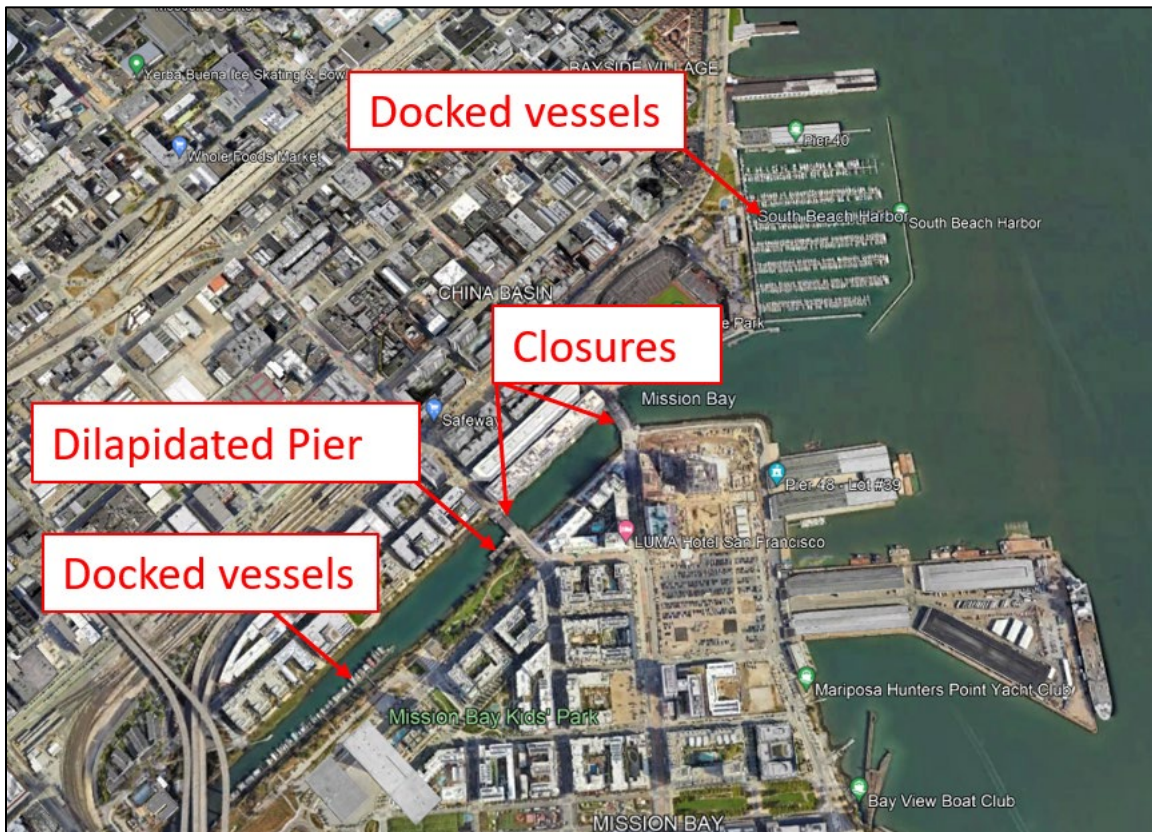


Figure 79. Potential debris sources and closure location (Google).

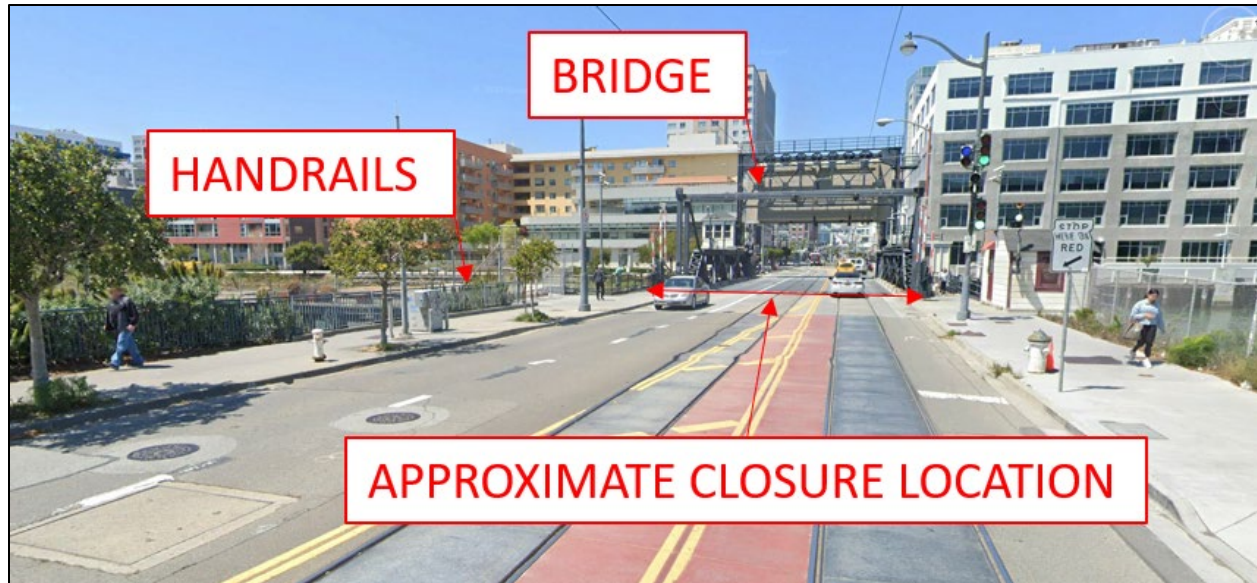


Figure 80. Closure location and impact impediments (Google).

A.67. PFM 67: Pier walls around wharves, which become line of protection after second action, fail (structural, stability, overtopping)

The team wanted to acknowledge that, if the pier walls being designed to reduce the risk of flooding to the wharves are adopted as the future line of protection, there are increased consequences if they were to fail. At a point in the future where a second action is being designed, these pier walls should be analyzed or modified to ensure risk is low.

A.68. PFM 68: Settlement and/or wave erosion of coarse beach EWN leads to ineffective wave reduction and wave overtopping breach occurs

The Engineering With Nature plan to restore coarse beach areas and vegetation adjacent to levees should ensure that a minimum levee prism is being maintained, even if some amount of settlement or erosion occurs. Additionally, if the EWN plan is being used to justify reduced wave energy for the levee design, these features need to be maintained effectively throughout the project life.

A.69. PFM 69: Debris blockage of flap gate leads to backflow

Any culverts 36-inches or greater through the walls or levees will be required to have two means of closure for redundancy to prevent backflow to the landside. All culverts are expected to be closed, through an active gate (e.g., flap gate), passive gate (e.g., slide gate), or both. If sediment debris builds up or woody debris gets lodged in the pipe, this debris could affect the gate closure. However, typically, once flow ramps up in a pipe it is typically self-cleaning in that it will move sediment and other debris out of

the pipe under high enough velocity. These culverts should be routinely inspected and the gates should be regularly maintained to ensure the function properly during high water events.

A.70. PFM 70: Failure of San Francisco Transition Structure (SFTS) leads to backflow through tunnel and flooding of interior (due to ship impact, seismic, etc.)

The San Francisco Transitions Structure (SFTS) is a structure that transitions the underwater Transbay Tube to the twin bored tunnel in San Francisco. The structure is located waterside from the proposed alignment. If the structure were to fail or overtop, flooding into the tunnel could result in backflow to the project area. This scenario is unlikely to occur and the resulting consequences would likely be limited to damages and life loss within the tunnel. The short duration of high storm tides would lead to limited backflow into the project area and any interior exits for water could be dealt with by constructing rings to contain the water.

Appendix B: Calculations

Appendix C: Review Documentation